# EXPERIMENTAL DETERMINATION OF THE RESIDUAL COMPRESSIVE STRENGTH OF CONCRETE COLUMNS SUBJECTED TO DIFFERENT FIRE DURATIONS AND LOAD RATIOS

by

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## Author's Declaration Page

I hereby declare that I am the sole author of this thesis. This is a true copy of the thesis, including any required final revisions, as accepted by my examiners. I understand that my thesis maybe made electronically available to the public.

# Abstract

The superior thermal property of concrete associated to its great capability of thermally insulating the embedded reinforcing steel rebar is the main reason for the good behavior of reinforced concrete structural elements in fire condition, compared to other construction materials. However, at elevated temperatures, concrete still undergoes changes in its mechanical and thermal properties, which mainly cause degradation of strength and may lead to the failure of the structure. Retrofitting is a desirable alternative to rehabilitate post-fire concrete structures. However, in order to ensure safe reuse of fire-exposed buildings and to adopt proper retrofitting methods, it is essential to evaluate the residual strength capacity of fire-damaged reinforced concrete structural elements.

The focus of this experimental research study is to investigate the fire performance of reinforced concrete columns exposed to elevated temperatures that followed CAN/ULC-S101 standard fire, and then evaluate their residual compressive strengths after fire exposure. In order to effectively study the fire performance of such columns, eight identical 200 x 200 x 1500mm high reinforced-concrete column test specimens were subjected to two different durations of standard fire exposure (1 hour and 2 hours) while being loaded with two different axial load ratios (20% and 40% of the column ultimate design axial compressive load capacity). In a subsequent stage and after complete cooling down, residual compressive strength capacity tests were performed on the fire-exposed columns. Experimental results showed that the residual compressive strength capacity dropped to almost 50% and 30% of its ambient temperature capacity for the columns exposed to 1- and 2-hour fire durations, respectively. It was also noticed that the applied load ratio has less effect on the column's residual compressive strength compared to that of the fire duration.

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**Nomenclature**

$P_{ro}$ = Factored axial load resistance

$A_g$ = Gross area of column cross section

$A_{st}$ = Total area of longitudinal reinforcement

$f_c'$ = Characteristic compressive strength of concrete

$f_y$ = Yield strength of steel

$T_f$ = Standard fire temperature

$T_w$ = Surface Temperature

$T_c$ = Concrete temperature at 'x' distance

$f_{yt}$ = Reduced yield strength

# Chapter 1       Introduction

## 1.1     Background

Fire is considered as one of the most devastating causes of structural damage along with other natural calamities, e.g., tsunami and earthquake. Fire design of concrete structures have conquered the attention of many researchers from time to time, since concrete is one of the primary construction materials and its inherited fire resistance has acquired a great deal of attention. Reinforced concrete columns are the most essential building components, and any local failure of columns may lead to the collapse of the entire structure (Balaji *et al.*,2015). Reinforced concrete columns are usually designed to perform at normal conditions with adequate safety factors under the presence of service loads. However, in fire condition, the response of the structure is unpredictable since fire is usually not considered like wind or earthquake loads during the design procedure of a building. Research initiatives around the world during the past few decades have helped in documenting the behavior of different reinforced concrete structural elements and assemblies under fire exposure (Bikhiet *et al.*, 2014). Fire safety measures in buildings include both active and passive fire protection systems. In active system, automatic devices are activated in the presence of smoke or at high temperatures, whereas passive fire protection systems make use of fire-rated building elements help in maintaining the structural integrity of such elements when exposed to elevated temperatures of fire. The capability of adequate concrete cover to thermally insulate the embedded steel reinforcement in a reinforced concrete structural element can be considered as one of the very effective passive fire protection systems.

Concrete is an extensively used construction material for various type of structures, e.g., buildings, bridges, retaining walls, water tanks, etc. In 1824 Joseph Aspdin, the English inventor, invented Portland cement which has remained the major form of cement used in concrete production. And the reinforced concrete was invented by Joseph Monier in 1849, who received a patent in 1867 for his invention. Concrete is a composite material that consists of main aggregates bound by a matrix of hydrated cement paste which hardens and become a solid mass like rock. Concrete gains strength very rapidly in the first few days after which the rate of strength gaining decreases steadily. Concrete is usually designed based on its $28^{th}$-day strength, by which time it acquires most of its strength. The main advantages of concrete constructions are cost-effective, sometimes

speed of construction or architectural appearance. Moreover, concrete is a good fire-resistant material due to its inherent non-combustibility and low thermal conductivity. All concrete structures are designed to perform well at ambient temperature even under extreme loading conditions. Good quality of concrete can be prepared by proper design for service conditions and it is much more durable against harsh environmental conditions compared to other construction material, such as steel and timber. Concrete can be made resistant to physical attacks, e.g., cycles of freezing and thawing and abrasion, as well as to chemical attacks, e.g., dissolved sulfates or acids or highly alkaline solutions.

## 1.2 Behaviour of Concrete at High Temperatures

Majority of fire hazards are caused by human errors. When concrete material is exposed to high temperatures it undergoes chemical changes, mineralogical changes, and changes in its water content. High temperatures cause deterioration which appears in two forms; local damage (cracks on the material surface) and global damage (failure of the element). The load-carrying capacity of a RC structure deteriorates as the temperature increases mainly due to the reduction of the modulus of elasticity and strength of both concrete and reinforcing steel. Spalling is an important phenomenon which affects the fire performance of concrete structures. At high temperatures, the pore water pressure increases due to the vaporization of moisture content in the cement paste. In concrete, when the tensile strength due to pore pressure reaches the maximum tensile strength of concrete, the cover may break off. Spalling causes decrease in the cross section and consequently reduction in the load carrying capacity of the concrete cross section. Since concrete cover split off, the reinforcing rebar become directly exposed to high temperatures and this causes significant reduction in the strength of the reinforcing rebar. Spalling is a complex phenomenon and several studies have been carried out by many researchers worldwide to better understand its causes and negative impact of concrete structural elements subjected to fire. High moisture content, rapid increase in cross-section temperatures and high slenderness ratios of concrete elements are the main factors which influence spalling. Also, high-strength concrete is more vulnerable to frequent spalling than normal-strength concrete due to its low pore volume which leads to high pore pressure (Shah and Sharma, 2017).

Latent heating is another phenomenon taking place in concrete when exposed to high temperatures. Due to the low thermal conductivity of concrete, it takes longer time to surge the temperature of core concrete. This means that heat does not dissipate quickly in concrete. Likewise, during the cooling phase, even though the surfaces of a concrete element reach ambient temperature, its core could still be at elevated temperatures. So, the concrete never regains its initial strength even after cooled down to ambient temperature. This non-uniform thermal propagation through concrete cross section results in unrecoverable strength (Dimia *et al.*, 2011).

At high temperatures, the chances of eccentric loading on concrete elements, such as columns, are high. The eccentricity is measured from the plastic centroid which is the location of the centroid of the resisting forces. At normal conditions, the plastic centroid is located at the centre of a symmetrically reinforced column. But, depending upon the condition of fire exposure, the plastic centroid shifts its position to either right or left of the original plastic centroid location. This centroid shifting leads to either increase or decrease in the eccentricity which can lead to additional bending moment on the cross section. Slenderness ratio of RC columns has considerable influence in fire condition. The slenderness ratio of a column is the ratio of effective length ($kL$) to the radius of gyration ($r$) of its cross section. The radius of gyration mainly depends on the cross-sectional dimensions and temperature gradients across the cross section. Accordingly, high temperatures can cause considerable increase in the slenderness ratio of fire exposed concrete columns. As a result, the chances of column failure due to flexural buckling when exposed to elevated temperatures of fire is higher than at ambient condition.

## 1.3    Residual Strength of Fire-Exposed Concrete Columns

It is rare that a concrete building would collapse ultimately after a fire incident mainly due to the concrete inherent non-combustibility and low thermal conductivity (Chen *et al.*, 2009). However, at high temperatures, thermally induced strains developed in concrete elements and cause considerable reduction of the concrete compressive strength and may lead to the failure of the structural element. The residual strength of fire-damaged concrete structural elements is an important factor in determining the feasibility of repair of such element. Building codes require the restoration of load-carrying capacity and fire resistance of a fire-damaged structure to certain acceptable levels. Repairing of fire-damaged concrete structures is more economical, both in terms of cost and time than demolishing and reconstructing such structure. However, after fire incident,

the assessment of the load bearing capacity of reinforced concrete structures is a complex task. This includes estimating the actual strength of concrete and reinforcing steel and the computation of the cross sections' capacities (Venanazi *et al.*, 2008).

The severity of fire exposure considerably affects the residual strength of RC structures. Fire exposure is influenced by fuel load, ventilation characteristics and geometrical parameters of the fire compartment. Another factor which affects the residual strength of concrete structural elements is the degraded mechanical and thermal material properties of both, concrete and reinforcing steel, which are also changing during the heating and cooling phases. Different studies show that there is up to about 10% reduction in the compressive strength of concrete after exposure to a temperature of only 220°C. Load level and restrained conditions of a RC structural element play an important role in its post-fire characteristics. To better understand the post-fire behaviour and accurately estimate the residual compressive strength of fire-damaged concrete columns, detailed analysis that considers fire exposure from different sides of the columns, spalling and creep effects, realistic fire exposure and different strain (mechanical, thermal, transient and creep) development is necessary. Alternatively, experimental testing of post-fire concrete columns to accurately determine its residual compressive strength.

## 1.4    Fire Resistance Testing

The current practice of fire resistance testing is to expose structural members to a standard fire such as ASTM E-119, ISO-834 or ULC/CAN-S101 and monitoring its behaviour. The time-temperature curve for all standard fires is nearly identical. It has a fast-starting with an equivalent burning temperature of 538°C that reaches within five minutes. Standard fire does not have a decay phase, instead it is ever-increasing with up to 1260°C after 8 hours. Fire resistance generally refers to the property of a material to withstand fire. According to NBCC (2015), the fire-resistance rating is the duration of time during which the material can adequately withstand the passage of flame and transmission of heat (NBCC, 2015). The fire-resistance rating is determined by exposing a building component to standard fire. The point at which the member can no longer withstand applied load is considered as the fire resistance of the member.

There are several factors which affect the fire-resistance rating of concrete columns, such as , concrete type, member dimensions, reinforcing steel, cover and column eccentricities. At high temperatures, the strength and modulus of elasticity of concrete reduce and coefficient of thermal expansion increases. As a result, creep and stress relaxations are considerably higher. Depending upon the type of aggregate, the compressive strength of concrete changes during fire exposure. According to NBCC (2015), there are two types of normal weight concrete; Type S concrete and Type N concrete. Type S concrete consists of coarse aggregates, granite, quartzite, siliceous gravel or other dense material which contain at least 30% quartz, chert or flint. Whereas Type N concrete consists of limestone, calcareous gravel, traprock, sandstone, blast furnace slag or similar dense material containing not more than 30% quartz, chert or flint. According to CAN/CSA A23.3-14, the low-density concrete is Type L40S which consists of sand as fine aggregate and any low-density material as coarse aggregate. Type N and L40S have superior fire resistance property than Type S concrete. Also, the fire-resistance rating is proportional to the member dimensions, as concrete elements with larger cross-sections suffer less concrete strength loss than those with smaller cross-sections for a given fire duration. Reinforcing steel also suffers from loss of strength at high temperatures. Hot-rolled reinforcing steel retains its strength longer than cold drawn reinforcing steel at elevated temperatures. The fire-resistance rating of the reinforcing steel depends on the type of steel, maximum temperature attained, and the level of stress required in the steel. Nevertheless, concrete cover plays an important role in protecting the reinforcing steel embedded inside the concrete cross sections from elevated temperatures of fire. Hence, adequate concrete cover must be provided depending upon the conditions. A typical fire resistance test conducted on a simply supported RC column involves compression loading that is applied before the fire exposure and maintained throughout the entire duration of the fire test while temperatures are increased following the standard fire time-temperature profile. Normally, visual observations are made during the fire test and cross-sectional temperature measurements, axial deformations, strain development are monitored.

## 1.5    Research Significance

All reinforced concrete structures are designed to perform at normal conditions with considerable factors of safety. Generally, concrete structures exhibit good fire behavior mainly due to the low thermal conductivity of concrete as a construction material. Reinforced concrete columns form

the main load-bearing member in a building. Hence, appropriate fire safety measures for reinforced concrete columns is very crucial to guarantee adequate building fire safety (Kodur *et al.*, 2004). There are several factors that influence the extent of damage in a fire exposed reinforced concrete column, such as fire severity, recovery time, temperature-induced steel-concrete bond degradation, restraint conditions, etc. (Hibner, 2017). Accordingly, it is important to evaluate the effectiveness of parameters such as load ratio and fire duration on the residual compressive strength of reinforced concrete columns. The residual strength evaluation is a complex task but essential before proceeding with any retrofitting procedure to be undertaken in order to rehabilitate and ensure safe reuse of fire-damaged concrete structures.

## 1.6    Scope and Objectives

In the study presented in this thesis, the residual compressive strength of fire-exposed concrete columns was evaluated through experimental testing to better understand the effect of different load ratios and fire durations on the post-fire performance of RC columns. This exercise provides valuable knowledge required for establishing the relationship between various parameters which affect the residual compressive strength of such fire-damaged columns. A total of ten identical medium-scale column specimens were prepared. Two specimens were tested at ambient temperature until failure and were used as control specimens to determine the column's ultimate failure capacity; whereas, the other eight specimens were exposed to elevated temperatures that followed CAN/ULC-S101 standard fire. Afterwards, the fire-exposed columns were tested under concentric axial loads until failure to determine its residual compressive strength.

The main objectives of this unique experimental study are listed below;

1. Design and fabricate reinforced concrete columns for fire resistance testing;
2. Study the structural behaviour of RC columns under standard fire exposure;
3. Evaluate the residual compressive strength of fire-damaged RC columns;
4. Investigate the effect of parameters such as load ratios (20% and 40% of the columns' ultimate design capacity), and fire durations (1-hour and 2-hour) on the residual compressive strength of fire-exposed RC columns.

# Chapter 2    Literature Review and Background Research

## 2.1    Introduction

Concrete is considered as a good fire-resistant construction material because of its natural non-combustibility and low thermal conductivity. Concrete has been widely used in buildings and civil engineering projects because of several factors such as cost, speed of construction or architectural appearance. Each year, over five million of fire events are registered all around the world and caused the death of 50,000 people and billions of property losses. Few reinforced concrete structures undergo catastrophic failure by the heavy and lengthy fire. However, some reinforced concrete structures that exposed to fire and still retained some of its strength. Those structures can be put back into use by retrofitting.

This thesis is focused on study of reinforced concrete columns only. When concrete columns are exposed to fire, it experiences material property changes due to high temperature. The overall strength of the column decreases because of the decrease in yield strength and modulus of elasticity of steel and concrete. They reach the limit state in relatively shorter time under fire because of the small size. Under fire exposure, the behaviour of constituent elements such as concrete or reinforcing steel are complicated and the fire resistance is influenced by the combined effect of dead loads, live loads and thermal effects. At high temperature the mechanical properties of concrete and reinforcing steel become temperature dependent. The thermal properties such as thermal conductivity, specific heat and density which are required for heat transfer analysis, also become functions of temperature.

Reinforced concrete columns must satisfy only one criterion which is their bearing capacity. The element must retain its constructive stability under fire for a specified period of time in order to maintain structural integrity. A series of complex processes starts to develop in a reinforced concrete column when they are exposed to fire for long durations. The fire resistance of reinforced concrete column is affected by number of factors;

- Dimension, cross section and Effective length of element
- Boundary conditions and Types of aggregates
- Concrete cover and total area of reinforcement and concrete

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- Mechanical characteristics and humidity of the concrete
- Loads- axially loading, uniaxial or biaxial bending
- Nature of fire exposure- standard fire or design fire

## 2.2 Concrete

Concrete is the most commonly used construction material which gives unlimited opportunities for different forms of constructions. Concrete is known as a universal material as it is composed of fine aggregate, coarse aggregate, water and cement that hardens over time. Properties of a concrete can be changed by altering any of the ingredients.

Concrete has versatile properties of fire resistance, durability, less maintenance, on site fabrication, energy efficiency, aesthetic properties, ability to cast, and economical. The disadvantages of concrete include law tensile strength, low ductility, volume instability, and low strength to weight ratio. Manufacturing of concrete has several steps such as, proportioning, batching, mixing, placing, compacting, finishing and curing. A fresh concrete can be moulded into any shapes. Concrete has high compressive strength whereas the tensile strength is very low. To overcome this, concrete is used with steel reinforcement.

Water is the key ingredient in the manufacturing of concrete which act as the lubricant for fine and coarse aggregate. A good quality of concrete can be produced from good quality of water. The water should be free from all impurities such as suspended or dissolved solid particles, organic materials etc. Impurities in water affects the setting times, drying, shrinkage, durability or they cause efflorescence.

70-80% of concrete consists of aggregates. The aggregates are granular materials derived from natural rock. Aggregates provide concrete a better dimensional stability as well as wear resistance. Even though aggregate strength has important role in high strength concrete, the strength of concrete and mix design are independent of aggregate strength. Natural sand is commonly used as fine aggregate. It should be free from clay, silt and all kind of impurities.

Water-cement ratio is an important parameter to be considered to produce strong and workable concrete. It is the ratio of volume of water mixed with cement. An optimum amount of water is required for a given proportion of materials which gives the greatest strength. Similarly decrease

in optimum water content also decrease the strength. The use of excessive water not only cause the low strength of concrete but also it increases the shrinkage and decrease the density and durability.

### 2.2.1    Plain Concrete

Plain cement concrete consists of cement, sand, aggregate, water and admixtures. The main properties of concrete are (a) strength, (b) durability, (c) workability and (d) economy. CSA defines the plain concrete as a concrete that contains no reinforcing or prestressing steel that the specified minimum for reinforced concrete.

### 2.2.2    Reinforced Concrete

Reinforced concrete makes use of both concrete and steel and hence its efficiency can be increased. In a RC structure the compressive force is taken by concrete and the tensile force is taken by the steel. The reinforced concrete is designed on the assumption that the two materials act together in resisting forces.

### 2.2.3    Prestressed Concrete

Prestressed concrete is a type of reinforced concrete in which the tensile stresses due to the external loads are taken by the internal compression stresses. High strength steel tendons are embedded within the concrete and using jacks the concrete is subjected to tensile stresses. Different types of tendons are wires, cables, bars, rods and strands. There are two methods are used for prestressed concrete; pre-tensioning and posttensioning.

Pre-tensioning: Tendons are tensioned before the concrete has hardened.

Posttensioning: Tendons are tensioned after the concrete has hardened.

## 2.3 Reinforced Concrete Columns

Reinforced concrete columns are the structural elements designed to carry compressive loads. Columns act as the vertical supports to beams and slabs and transmit the loads to the foundation. Reinforced columns consist of reinforcing bars and concrete. The main failure modes of columns are compression or crushing failure and buckling failure. Crushing failure occurs in short columns whereas buckling failure is associated with long columns. In all reinforced concrete structures, the compressive stresses are taken by concrete and the tensile stresses are taken by steel reinforcement. Columns are designed to support compressive loads with or without bending. The magnitude of bending moment and axial force determine the column behaviour and it will vary from pure beam action to pure column action. Columns are usually reinforced with longitudinal reinforcement and transverse reinforcement. Since the failure of columns leads to the complete damage of the structure, they are designed with high factor safety than beams.

### 2.3.1 Types of Columns

Columns can be classified as short or long columns, braced or unbraced depending dimensions and reinforcements. A column is considered as short when the slenderness ratio (ratio of effective length to its least lateral dimension) is less than 12. If the ratio exceeds 12, then it is known as long columns.

Columns are again classified based on the reinforcement.

1. Tied column: In ties columns the longitudinal bars are tied together with the transverse bars(stirrups) spaced at certain interval along the column length. These ties help to hold the longitudinal rebars in place during construction and ensure stability of these bars against local buckling. Usually square, rectangular and circular cross sections are used. A minimum of four bars is used in rectangular and circular cross sections.
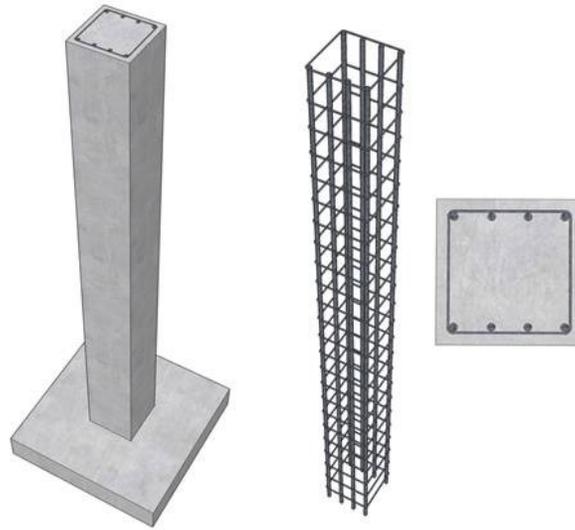
**Figure 2.1** Tied columns (Adapted from Structure point.org, 2018)

2.  Spirally reinforced column: If transverse reinforcements are helical hoops, the column is called spiral column. The longitudinal bars in these columns are arranged in a circle surrounded by a closely spaced spiral. A minimum of six bars are used and the cross sections are usually circular or square.
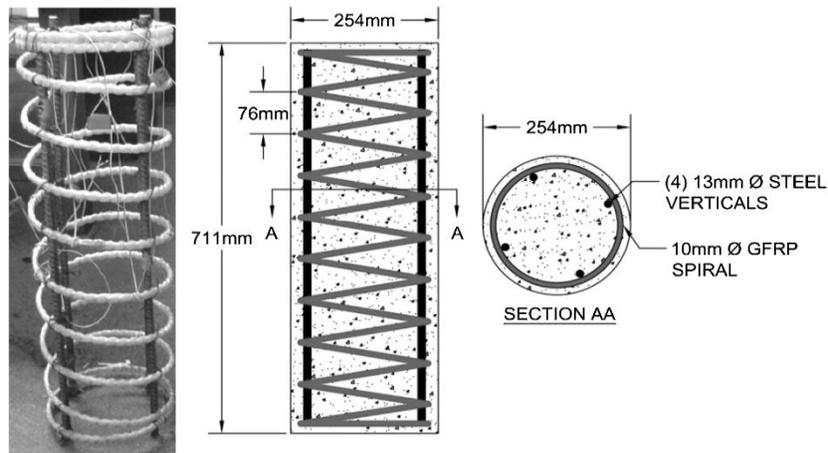


**Figure 2.2** Spirally reinforced columns (Adapted from ASCE library, 2018)

3.  Composite columns: Composite columns are made of structural steel shape or pipes surrounded by or filled by concrete with or without longitudinal reinforcement.

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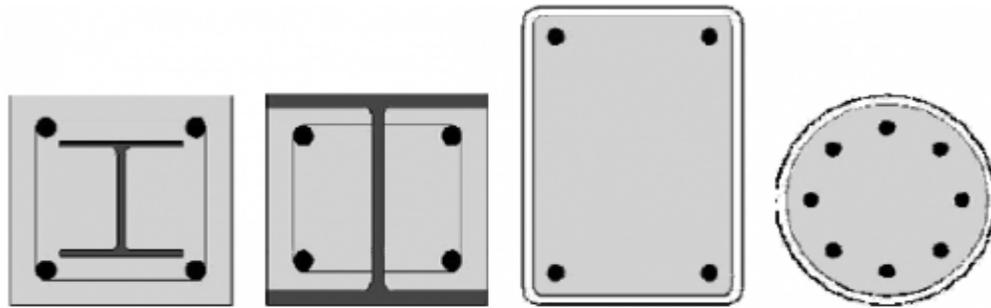
**Figure 2.3** Composite columns (Adapted from steelconstruction.info, 2018)

The failure in tied column occurs due to the excessive cracking in the concrete section followed by the buckling of the longitudinal reinforcement between the ties within the failure region. But in spirally reinforced column, when the ultimate load is reached the concrete shell covering the spiral starts to peel off. Then the spiral provides a confining force to the concrete core and thus enabling the column to sustain large deformations before the final collapse.

## 2.4 Behaviour of Reinforced Concrete Columns at Ambient Temperature

It is important to study column behaviour under normal conditions, in order understand the behavior of columns under fire. Columns are the main load carrying element which supports all other building structures for example bridge decks, floor slabs and can act as piers or piles. Thus, the column failure may lead to the failure of entire structure. A column failure can happen in two modes; crushing failure and flexural buckling. Crushing failure occurs when the applied force is greater than the axial capacity of column whereas, the flexural buckling occurs when the applied moment on the column is greater than the moment resisting strength of the column. Columns with low slenderness ratio usually prone to crushing failure. The concrete is brittle in nature. In common practise the reinforced concrete columns are designed to perform well in normal temperature. The several factors which affect the RC column capacity are longitudinal and transverse reinforcement, level of loading, spacing of ties and type of concrete. But also, the environmental factors such as earthquake, saline conditions, corrosion etc. affects the performance of concrete structures. During earthquakes, the damage of buildings or infrastructures leads to the huge loss of both human life and property. Aging of structures is also a major concern regarding the service life of the structures.

An early experimental study conducted by Green *et al.* (2006) to analyse the column behaviour under extreme conditions, like freeze and thaw actions, corrosion and fire. In order to improve column axial capacity, they wrapped columns with fiber reinforced polymer (FRP). The use of FRP materials in construction industry has started to increase since it can be easily applied for the rehabilitation and strengthening of RC structures. By wrapping FRP on reinforced concrete columns, the axial strength, shear strength and seismic capacity can be improved. The columns of bridges, parking structures are always exposed to freeze-thaw cycles and potential corrosion due to de-icing salt. The experimental results indicate that the columns which are subjected to low temperature, failed in a brittle manner. The FRP wrapped columns exposed to freeze and thaw cycle (16h of freezing at -18°C and 8h of thawing at +15°C) are failed suddenly and the wrapping helped only to hold the cylinder together. FRP wraps are effective for the corroded reinforced concrete columns as well.

The performance of RC columns is also affected by the eccentricity of applied loads. Li *et al.* (2018) have studied the eccentric compressive behaviour of RC columns which are strengthened using steel mesh reinforced resin concrete (SMRC). The effect of number of layers of steel mesh on the failure mode, cracking load and load capacity are the parameters studied and they have also conducted finite element analysis to evaluate the effect of reinforcement layer thickness and the load capacity. In the experiment, the resin concrete was manufactured from 3 ingredients, liquid epoxy resin, liquid curing agent and a mix of ordinary Portland cement and sand. For the experiment, columns were casted with different number of steel mesh layers, reinforcement ratio and reinforcement position. The SMRC application has shown in Figure 2.4.
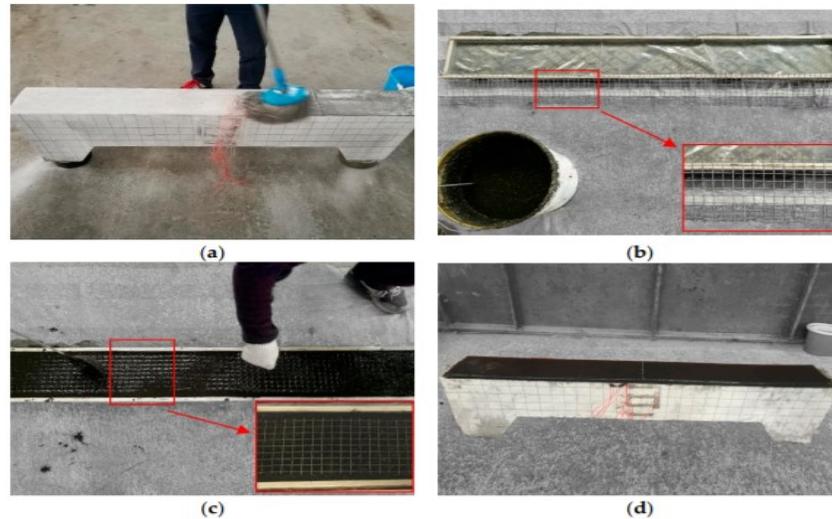
**Figure 2.4.** Strengthening process: (a) clean and wet surface; (b) installing formwork; (c) installing steel mesh; (d) cast resin concrete (Adapted from Li *et al*., 2018)

The load applied with an eccentricity of 150mm. And the results indicate that there was no debonding observed at the interface between concrete and SMRC. The cracking and load carrying capacity seem to be effectively improved and this method can be applied for rapid strengthening processes.

The effect of eccentricity on concrete columns was also studied by Hadi (2005). The increase in eccentricity decreases the strength capacity of columns. In the experimental program, they have tested concrete circular columns of 1.4-m high and 150-mm diameter. At each end of the columns were provided with haunches at one side. The concentrated load applied on the haunched ends so that it will create an eccentricity of 42.5mm on the test area. The FRP application as external confinement to the concrete structures gained the column strength and the moment resistance is also enhanced. The numbers of layers of FRP wrapping is also affect the strength as they absorb eccentric loads with increase in lateral deflection.

### 2.4.1 Experimental Testing of Reinforced Concrete Columns at Ambient Temperature

Compression column elements potentially support building structures like floor slabs, beams, bridge decks etc. The analysis of reinforced concrete column is important but there are a lot of researches already done. Nowadays researches are going on to find methods to improve the performance of concrete columns and structural retrofitting. Since the column failure leads to the

total collapse of the whole structure, column strengthening is a really important aspect in a building structure.

To improve the structural safety, the concrete columns were modified with high performance fiber reinforced cementitious composite and tested under normal conditions (Cho *et. al.*, 2018). Such concrete columns with strengthening bars have improved seismic performance and cyclic load carrying capacity as well. The old RC columns were modified by grooving on the surface and placed longitudinal and transverse strengthening bars and sprayed the HPFRC mortar. The results indicate that the columns were attained sufficient load carrying capacity and deformation capacity compared to conventional RC columns.

An experimental research done by Fan *et al*. (2016) has made use of inorganic polymer concrete with basalt FRP bars to cast concrete columns. The inorganic polymer concrete (IPC) together with basalt reinforcement has improved properties such as corrosion resistant and fire resistance. In order to reduce the carbon dioxide emission associated with concrete and to manufacture environmentally friendly binder inorganic polymer concrete can be used. It is manufactured from fly ash and ground granulated blast furnace slag. The IPC has good qualities such as little drying shrinkage, low creep, fire resistance, resistance to freeze-thaw cycles and acid attack. Basalt FRP (BFRP) also has good properties such as cost effective, ease of manufacturing, high temperature resistance, resistance to freeze-thaw cycles, corrosion resistance and resistance to acid and vibration and impact loading. In this research the mechanical properties of short IPC columns to that of Ordinary Portland Cement Concrete (OPCC) column have studied. The results show that under larger eccentricities the IPCC specimens have same load-displacement performance that of OPCC. But under low eccentricity the IPCC columns have 30% lower load carrying capacity than OPCC. (Fan *et al.,* 2016).

A paper by Adam *et al*. (2007) has studied the behaviour of axially loaded RC columns strengthened with steel angles and strips. The specimens were 2500mm long and 300X300 mm cross sectional dimensions. They have used concrete heads of 300X300X600mm at both ends for simulating column-beam connection. The concrete use for head and column were different and the compressive strengths were 90MPa for column head (HSC) and for column the compressive strength ranges between 10.6 to 15.5MPa. The reinforcement consists of four 12mm diameter

longitudinal roads and 6mm diameter stirrups at every 0.20mm. The angles L80.8 and rectangular strips 270X160X8mm were used for strengthening the columns. Three types of specimens were tested, called Test, A and B. The specimen 'Test' was non-strengthened column, specimen 'A' doesn't have connection between strengthening elements and head and specimen 'B' has connection between strengthening element and head. Instrumentation includes 14 strain gauges mounted on steel strengthening and 8 LVDT s to measure lateral and axial strains. The result indicates that in specimen A, there is an increase in compressive strength at ends due to the confinement caused by steel strips. This strip confinement effect can also be studied from the tensile stresses of metal angles resulting from cross sectional variation. This stress is considerable at end strips compared to middle strips. But in specimen B the strip confinement and strip tensile stressed are negligible. This paper has also studied the load transmission by shear stress at the interface bond between the cement mortar material and the strengthening steel. This depends on the coefficient of friction ($\mu$). Increase in $\mu$ value results better load distribution between concrete and strengthening steel. In specimen A, mainly there are two effects which affect the behaviour of strengthened column; they are strip confinement pressure and interface bonding material influence. It is recommended to reduce the strip spacing at the end of columns as failure always occur at the ends. The confinement effect will increase as the strip spacing decreases and thus the column performance can be improved. (Adam *et al.*, 2007)

In 2012, Cho *et al.* had also done an experimental research on HPFRC mortar. The plastic hinge regions of concrete columns were strengthened using HPFRC mortar. Plastic hinge regions are the vulnerable positions where brittleness and cracking occur and also the lack of lateral load carrying, and deformation capacity cause the flexural cracks, yielding and buckling of longitudinal rebars as well as the crushing of concrete in plastic hinge zone. This method can be useful for seismic strengthening. A set of columns were casted and tested under cyclic lateral load combined with axial loading. These columns have attained improved cyclic loading capacity as well as reduced the shear cracks.

## 2.5    Behaviour of Reinforced Concrete Columns at Elevated Temperatures

All the structures are exposed to multi hazards such as earthquake, tsunami, fire hazards etc. But fire hazard is the most vulnerable because it affects both mechanical and thermal properties of the

structure. Past histories such as Grenfell tower fire (London) on June 2017, fire attack in World Trade Center (USA) on September 2001, terrorist attack in Taj Hotel (India) on November 2011 etc. gives an overview of fire effects. The fire safety in building has an important role in concrete designing. In order to maintain the overall stability of the building, the concrete elements must have adequate strength to withstand the fire exposure. Since concrete columns are the main load bearing members in the buildings, the study of fire resistance of columns has significant importance.

All the structural members behave same as per the designed criteria at ambient temperature. But a fire scenario leads to significant changes in the mechanical properties of materials and reduction in strength of the structural member. Fire is the most destructive force that often causes death, injuries, billions of property losses etc. It is very difficult to predict the behavior of fire. This unwanted fire has become one of the greatest threats to all structures. Studies on fire performance, prevention and precautions for structural elements are still going on. There are two types of fire protection systems, active and passive. Active fire protection system activates only during fire situation (sprinklers, smoke detectors etc.) whereas passive fire protection systems make use of fire-resistance-rated walls, doors, windows that prevent the fire spreading quickly and provide time to fire fighters to perform evacuation processes.

Concrete structures are usually designed to perform at room temperature and fire safety check is done as per code provisions. But during unexpected situations such as explosions, unpredicted fire accidents, structures are exposed to high temperatures about $1000^0$C. Study of fire resistance of columns is important because columns are the main load bearing member in buildings. Columns should have adequate strength to withstand the effect of fire exposure. Fire resistance is defined as the duration during which a structural member exhibits structural integrity, stability and temperature transmission under fire conditions.

The common after effect of fire is spalling. Spalling is the concrete cover breaking off during the fire exposure. The moisture content in the concrete starts to vaporize at high temperature and which will increase the pore water pressure. This results the concrete cover splitting off and eventually decrease the cross-section area as well as overall cross section strength. The spalling is affected by high moisture content, rapid increase in cross-section temperature, high slenderness ratio, type of aggregate, stress conditions, porosity and permeability. High temperature exposure in a concrete

column causes the shifting of plastic centroid. It is the location of centroid of the tension, compression and resistance forces. This change in location of plastic centroid results either increase or decrease in eccentricity. Slender columns are more vulnerable to fire than short columns. Slenderness ratio is the ratio of effective length to radius of gyration. Since the radius of gyration depends on the column's cross-section, the temperature gradient has an influence on it. Hence slender columns are more susceptible to flexural buckling during fire exposure. Another phenomenon takes place in concrete when it is subjected to natural fire curve is latent heating. Due to the low thermal conductivity of concrete, it takes long time to increase the temperature of core concrete. During cooling phase, even though the outside temperature is decreasing the interior of the concrete could be still increasing. The outer surface cools down first than the core region of concrete. This leads to a serious issue because concrete does not regain its strength even after cooling down. Because of the phenomenon of latent heating, the interior of concrete could be still losing unrecoverable strength even when externally everything appears to be safe. (Dimia *et al.*, 2011)

### 2.5.1    Microscopic Study

Guruprasad and Ramaswamy (2018) have carried out a research at Indian Institute of Science Bangalore, on microscale properties of concrete and reinforcing steel which are exposed to high temperature. The results from micro analysis are useful to assess the extent of damage and its nature in terms of stiffness reduction. Microscale analysis can be used for the continuous investigation of properties such as elastic modulus, creep or micro hardness. It was already found that cementitious materials are complex, heterogenous and have a random micro structure using microscopic techniques like scanning electron microscope (SEM), transmission electron microscope (TEM) and image analysis. So, when concrete or steel is exposed to high temperature, the reduction in stiffness and strength at micro level can be analysed and interpret in macro level. The nature and extend of damage in concrete or steel due to high temperature is expressed in terms of stiffness reduction.
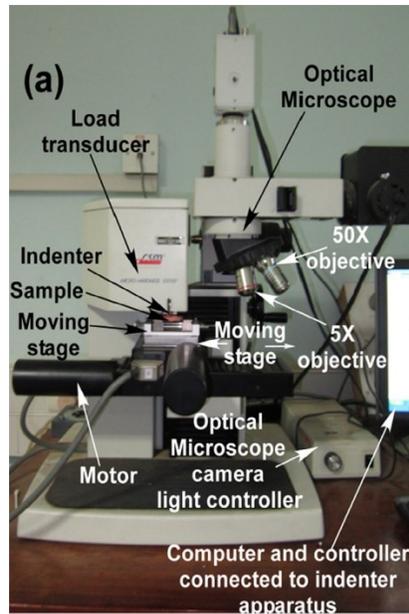
**Figure 2.5** Micro indentation apparatus

(Adapted from Guruprasad and Ramaswamy, 2018)

The samples are collected from heat exposed (different time and temperature) concrete cylinders. 25mm X 15mm X 15mm chunks of concrete were cut by using diamond saw. Similarly, a piece of 15mm steel rebar from heat exposed rebar was also collected. The results from this study are really useful in designing structural repair for concrete structures.

A similar research to analyse temperature history in concrete in microscopic level has been done by Annerel and Taerwe (2009) to determine the maximum temperature. They have used three methods for this test. (a) scanning electron microscopy to study the physico-chemical transformation (b) polarising and fluorescent microscope and (c) influence of heat on the colour of aggregate. Since concrete behaves well in fire, the temperature history analysis would help to repair or strengthening the structure. This would impart economic benefits such as cost of rebuilding or demolishing and also the building could be reused even faster. Concrete undergoes chemical and mineralogical changes by heating which may alter the colour of the concrete, composition of cement pastes and its porosity. Microscopic analysis can be effectively used to track the visible changes. The colour of concrete changes as increase in temperature; red at temperature between 300°C to 600°C, whitish gray at 600°C to 900°C and buff at 900°C to

1000°C. Reddish colour of concrete is because of the oxidation of iron hydroxides in the aggregates and the cement paste. To repair this, the common practice is to cut the concrete to the depth of red colour. The whitish gray discolouration is related to the disintegration of calcareous constituents of aggregate and cement paste. This paper has studied mainly the colour changes in siliceous and calcareous aggregates after heating up to 1150°C. Aggregates also undergo physico-chemical degradation and result the change of colours.
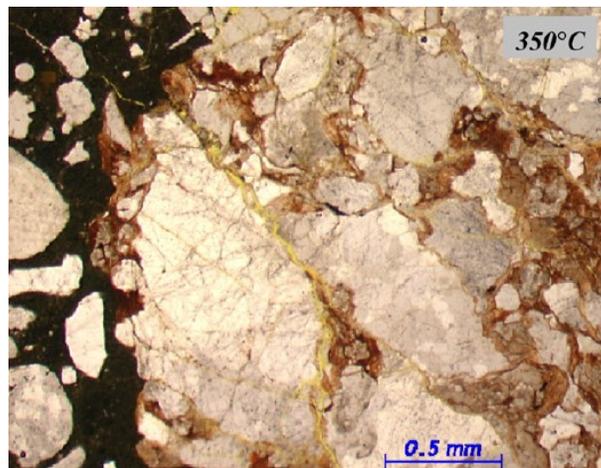


**Figure 2.6** Reddish brown coloration of siliceous aggregate at 350°C (Adapted from Annerel and Taerwe, 2009)

The reddish tint in the siliceous gravel is due to the oxidation of iron at 300°C. However, the color keeps on altering as the temperature increases. This colour change is really useful to determine the temperature history of concrete.

Cracks are the noticeable changes occur in heated concrete. There are two types of cracks; cracks around the perimeter of course aggregate (interfacial cracks) and cracks in the cement matrix (cement matrix cracks). While considering structural elements, the paper says that when a loaded concrete is heated, the strength reduction is less, and this is because of Load Induced Thermal Strain (LITS). This LITS happens only during first time heating and mostly important for columns. Cracks in concrete element occur due to the combined effect of the increase in pore pressure and the external loading. In fire if the thermal expansion of concrete is retrained, it may lead to the shear failure of columns. Also, the internal cracks develop due to the displacement difference between inner cold concrete and the heated outer layers which results tensile stresses.

## 2.5.2 Experimental Testing of Reinforced Concrete Columns at Elevated Temperatures

The high temperature exposure on concrete column leads to the strength reduction of reinforcement as well as concrete. A recent experimental study conducted by Shah and Sharma (2017) to investigate the effect of concrete strength, confining reinforcement and the configuration of confining reinforcement on fire resistance of reinforced concrete columns. According to Shah and Sharma the fire resistance has defined us the duration during which a structural member exhibits resistance with respect to the structural integrity. The fire resistance rating in building codes are usually based on the size of members and the cover to the reinforcement while, factors such as load ratio, fire exposure, material strength, slenderness ratio and reinforcement are not considered.

For the experiment eight full scale reinforced concrete columns with different confinement and material, are exposed to standard fire. The study shows that high strength concrete has lower fire resistance and explosive spalling while normal strength concrete has higher fire resistance. In concrete increase in moisture content increases the risk of spalling. All the columns are tested for fire resistance under constant concentrated loads. Columns are loaded 1 hour before the test. The temperature in the NSC is lower than that of HSC because of difference in thermal properties and the higher compactness (lower porosity) of HSE. The lower porosity of the HSC affects the rate of increase of temperature until the crack widens, and spalling occurs.

The temperature in less confined columns are higher than more confined columns. This may lead to the spalling in less confined columns. All the columns failed when the rebar temperature reached $750^0$C. In highly confined columns, the ties protect the core concrete and increased the time to failure. The spalling pattern changes with the confinement levels. The HSC columns though susceptible to thermal spalling can be safeguarded by using extra confinement. Spalling results reduction of cross section and reduces the fire resistance since the rebars get exposed to fire directly. Cross ties help to increase the fire resistance because it holds the longitudinal rebars firmly in place. 100% increase in confinement increase the fire resistance by 90 minutes in NSC columns. The confinement of columns has importance in fire resistance. The effect of confinement is more significant in NSC columns than HSC columns as 50% increase in confinement increases the fire resistance by 12% and 3% in HSC columns.

Another experimental research was conducted by Hana *et al.* (2013) on fire testing of composite columns. The columns were square cross sections cross-shaped steel and H-shaped steel. It was found that the fire resistance of all the specimens are above 150 minutes. They investigated the fire resistance of steel reinforced concrete columns with various section types and load ratio above 0.5 and subjected to standard fire ISO 834. The failure occurred mainly due to lateral flexural buckling and local concrete crushing. The load level and slenderness ratio clearly affect the fire resistance whereas the factors such as steel ratio, load eccentricity ratio, reinforcement ratio, strength of concrete etc. have less effect. They concluded that composite columns have excellent fire performance because of the "composite action" between the encased steel and the peripheral concrete.
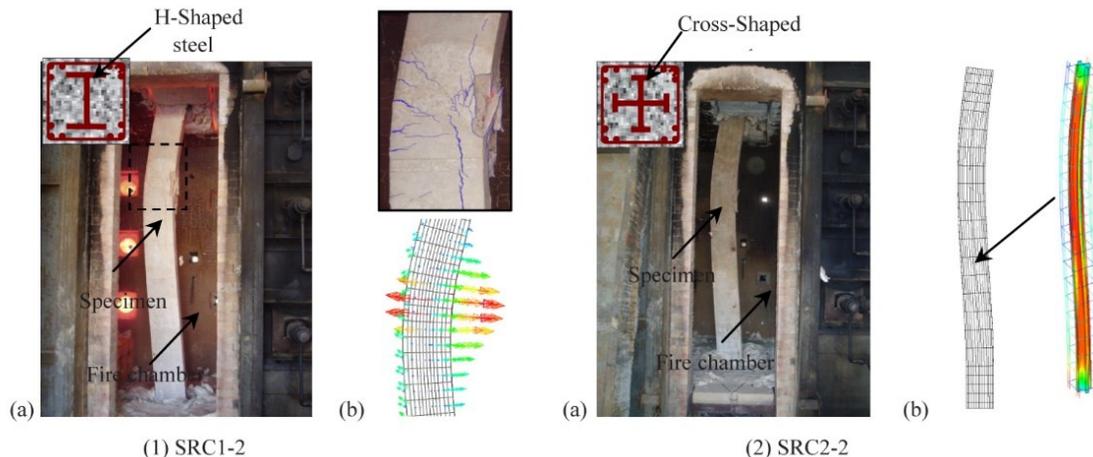


**Figure 2.7** SRC column (Adapted from Han *et al.* 2013)

Martins and Rodrigues (2010) have conducted experimental and numerical research to study the behaviour of RC columns under fire with restrained thermal elongation in the University of Coimbra, Portugal. Their study examined longitudinal reinforcement ratio, slenderness of the column, and the stiffness of the surrounding structure to the column in fire. The test specimens were RC columns of 3m long and two different cross sections (250mm x 250mm and 160mm x 160mm). Three different sizes of rebars were used for longitudinal reinforcement in different columns (10mm,16mm and 25mm). A load of 70% of column's design load was maintained throughout the test. The concrete columns restrained by attaching to the frame. The connection

between the concrete columns and the steel end plates were strengthened by welding the longitudinal steel reinforcement bars and a steel hook to them before concrete casting. The effect of longitudinal reinforcement ratio and slenderness were same as that of other authors stated. They have also found that, there is a reduction on fire resistance when the columns are axially restrained. A similar experimental research was also performed by Ali *et al.* (2010) at Fire research laboratories, The university if Ulster. They have studied 30 high strength concrete columns under five different loading levels and 2 heating rates. The columns with cross-section 127 x127 x 1800 mm high, four 12 mm longitudinal rebar, 20 steel ties (6 mm diameter at 120 mm intervals at middle and 60 mm at ends) were subjected to two different heating rates as shown in Figure 2.8.
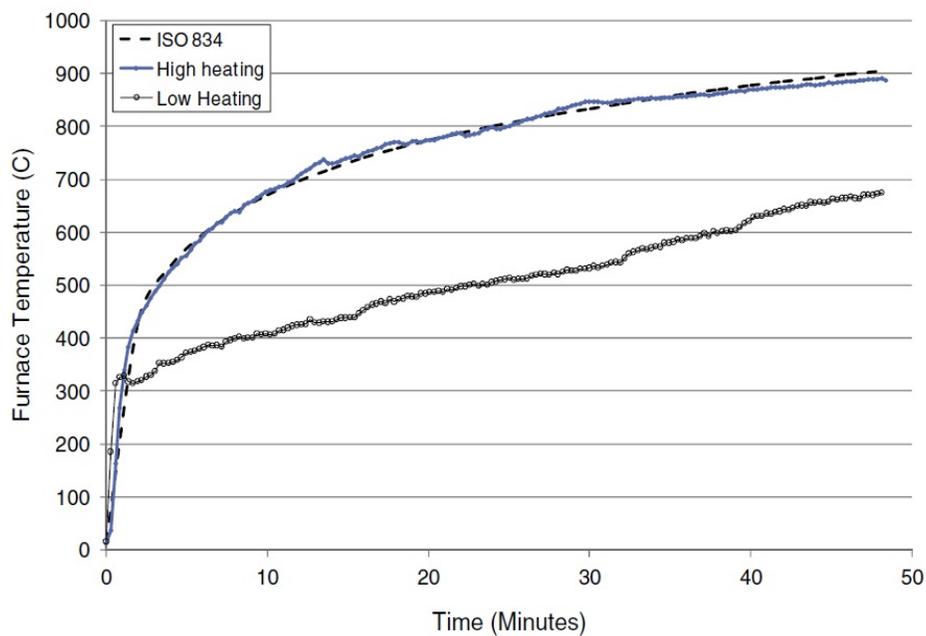


**Figure 2.8** Fire curve for experimental testing (Adapted from Ali *et al.*, 2010)

The high heating rate curve reaches 600 °C in 6 minutes whereas, low heating rate reaches it takes 40 minutes to reach 600°C. The results show that loading level has a clear influence on spalling. The fire resistance reduced by an average of 65% as the load level increased.

## 2.5.3    Factors Influencing Behaviour of Reinforced Columns in Fire

A numerical study was conducted by Balaji *et al*. (2016) on the behaviour of reinforced concrete short columns subjected to fire. Eight parameters are investigated in this paper: thermal boundary conditions, grades of concrete, grades of steel, types of aggregate, distribution of reinforcement on column faces, concrete cover, load eccentricity and support conditions. For the analysis, various cross-sections of columns were considered. Based on the results, parameters such as load level, amount of steel reinforcement, effective length of column, concrete strength, moisture content, area and shape of cross section, characteristics of fire exposure and aggregate type have significant influence on fire resistance. The strength reduction of steel and reduced cross-section of concrete was calculated with respect to the temperature obtained from thermal profiles using the reduction factor taken from EC2. After finding the reduced strength the procedure to calculate the axial capacity is same as that of normal design. According to Balaji *et al*. (2016) various parameters such as thermal boundary conditions, reinforcement type, reinforcement distribution on the surface of concrete, cover to reinforcement, type of aggregate and concrete strength have significant effect in altering the fire rating of column as explained below.

- ➢ Effect of boundary condition: The fire exposed surface area has significant importance in fire rating. Columns are constructed either separately or along with walls. Individual columns are usually exposed from all side whereas columns along with partition walls have different fire exposure.
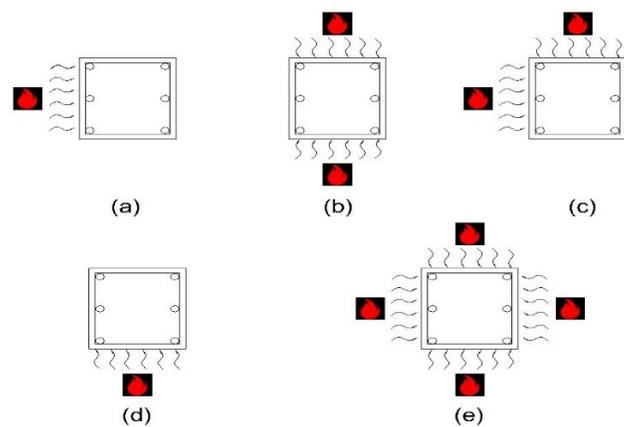


**Figure 2.9** Various surface areas of fire exposure (a) on single face; (b) on two opposite faces; (c) on two adjacent faces; (d) on half of the cross section; (e) on four faces (Adapted from Balaji *et al*., 2016)

This unsymmetrical exposure leads to unsymmetrical changes in the column and as a result plastic centroid shifts from its position and thus an eccentric loading occurs. As the eccentricity increases its fire resistance decreases with increase in restraints. This is because, the eccentricity induces an additional moment on the column which produces excessive lateral deflection and thus stiffness reduction occurs. This result loss of capacity of columns and causes failure.

➢ Fire exposure intensity: The rate of fire exposure significantly affects the fire resistance of concrete columns. The intensity of fire depends on several factors such as, fuel availability, ventilation and active fire protection systems.

➢ Effect of concrete cover: as the cover reduces the temperature on reinforcement increases which leads to the degradation of strength. Increasing the cover from 40 to 60 mm increases the fire rating by 80 minutes.

➢ Effect of concrete density: Light weight concrete has more fire resistance capability than normal strength concrete. It also related to the moisture content present in the light weight concrete which cause fire induced spalling due to increased ore water pressure.

➢ Effect of grade of concrete: The thermal properties of concrete are complex, but it depends upon moisture and porosity. Studies show that effect of spalling is not a major issue in normal strength concrete. Axial capacity of columns increases with increase in concrete strength and with increase in cross section. Fire resistance of high strength concrete is less due to the explosive thermal spalling. HSC columns with larger cross-sections show lower fire resistance than NSC columns.

➢ Effect of grade of reinforcement steel: During the initial stages, the axial load capacity increases with increase in grade of steel. But it decreases as the time of exposure is more. The steel reinforcing bars need to be protected from fire exposure. Steel with low carbon contents exhibit brittles at temperature between 200-300℃.

➢ Effect of distribution of reinforcement: Studied by distributing reinforcements on four sides and two opposite side. 2% is the reinforcement percentage in both cases. There is only slight improvement in both axial capacity and fire resistance for four-sided reinforcement than that with reinforcement of larger diameter distributed on two faces. It is inappropriate from a fire resistance standpoint to place a substantial amount of reinforcement in corners of the cross section. Instead of keeping the reinforcements in corners, it should be

preferably distributed on the faces. Also, an increase in reinforcement ratio leads to lower fire resistance. This is mainly because as the temperature rises steel losses its strength faster than concrete.

➤ Type of aggregate: Carbonate and siliceous aggregates are studied in this paper. Carbonate aggregate has slightly more strength and fire resistance than siliceous aggregate because carbonate aggregate has low thermal conductivity resulting in higher fire resistance.

➤ Effect of load intensity: Load intensity has significant influence on fire resistance. The fire resistance decreases with increase in load ratio.

➤ Concrete moisture content: High moisture content causes more spalling and reduce the fire resistance. Some fire tests showed that spalling is considerable when the relative humidity is higher than 80%.

## 2.6 Experimental Testing of Residual Strength of Concrete Columns

Residual strength index is defined as the ratio of the ultimate strength corresponding to the fire duration time ($t$) to that at ambient temperatures. The awareness of residual strength of concrete structure is very important to conduct an assessment and conclude if the structure should be demolished or retrofitted for the further use. The residual strength of concrete is widely affected by maximum temperature attained, mix proportions, cooling time, temperature induced bond degradation, restrained conditions and the loading conditions. Up to 100 ℃ there is no reduction in strength. But over 500℃ the about 50% of strength will be lost. There are only limited studies exist that explains about residual strength of concrete columns after fire exposure.

Kodur *et al*. (2017) have done a research on residual capacity of concrete column after fire scenario. The main parameters studied are temperature, spalling and deformations in the columns. This paper says that even after a fire exposure, it is very rare that the structure collapse completely. The concrete retains some of the strength because of the properties such as low thermal conductivity, high thermal capacity, and slower degradation of mechanical properties. Only the effect of spalling can be seen by visual inspection. For the reuse or retrofitting it is very important to find out residual strength. The experiment was conducted on two columns with two different

design fires and load ratios (50% and 55% of nominal capacities). Only middle 1700 mm of the column was exposed to fire.
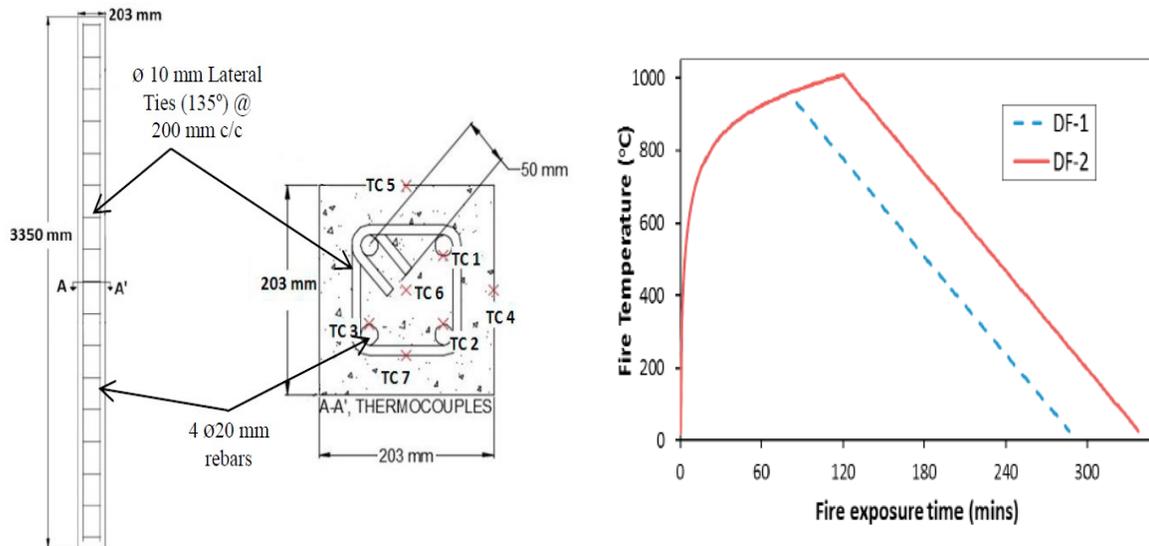


**Figure 2.10.** Reinforcement details and design fire curve (Adapted from Kodur *et al*., 2017)

The residual capacity was tested 48 hours after the heating phase, and then loaded until failure. The results show that, the residual capacity of concrete columns after 90 minutes and 120 minutes of fire exposure, ranges 34% and 29% respectively. The study concludes that, to improve the residual capacity a well-defined cooling phase has importance.

A similar experimental program was also performed by Kodur *et al.* (2013) on high strength concrete columns with the same dimensions as shown Figure 2.9. After completing cooling phase, they conducted residual strength testing. During this experiment 40 to 60 percentage of the design capacity maintained under three different heating phase and cooling phase. The results from residual strength testing shows that all five columns experienced un recoverable axial deformation during fire exposure. The columns with highest cooling rate experienced no spalling and retained almost 90% of the original load carrying capacity after fire exposure. This can be attributed to the partial regain of strength by concrete and steel upon cooling and strain hardening of the steel reinforcement. And it also concludes that the residual load carrying capacity of fire exposed columns depends on the cross-sectional temperature attained.

In 2013, Kodur *et al*. evaluated the residual strength and proposed a simplified approach. They have explained that the loss of strength of concrete columns after fire exposure depends upon different factors such as type of fire, column size, properties and temperature of steel and concrete and load ratio.

Another research was conducted by Hana *et al*. (2005) regarding the performance of spalling resistance of high-performance concrete with poly propylene fibre contents and lateral confinement. They have done an experimental research on fire resistance of concrete with poly propylene fibers and performed the residual compression test as well. The specimens were also provided with lateral confinement with metal fabrics, carbon fiber and glass fiber. The test result indicates that as the thickness of lateral confinement using metal laths increases residual compressive strength ratio also increased up to 80-90%. But for glass or carbon fiber confinement the residual compressive strength ratio was 30% only. Concrete with poly propylene fibers and metallic fabric confinement shows 90% of residual strength because of spalling resistance.

Bikhiet *et al.* (2014) conducted a research to study post-fire exposed columns. Fifteen column specimens of 15 x 15x 100 cm cross-section subjected to a temperature of 600°C. The parameters of this study include concrete strength, fire duration, load level, percentage of longitudinal reinforcement and yield strength, and bar diameter. During fire exposure all the columns experienced concrete cover cracking and spalling. After fire test, columns allowed to cooldown and then, tested the remaining strength of columns. During test, it was observed that the columns exposed to fire showed first cracks at a load level of 50% of column failure load due to the loss of stiffness. Crack pattern was vertical, and deformations occurred at the mid-height of the column. They also conducted a non-linear finite element analysis. The concrete characteristic strength affected the failure load after fire. The failure load increased 25% as the concrete strength increased from 300 to 500kg/cm$^2$). From both theoretical and experimental analysis, it is found that the column stiffens reduced as the fire duration increased. Also, the applied load during fire affected the failure load. The failure load decreased 3-6% when the load level increased from 10 to 20 tons. Both longitudinal reinforcement percentage and diameter have positive influence on failure load of post-fire exposed column. The method of cooling after fire exposure has a major effect on failure pattern. The column cooled in a room temperature has more stiffness than the column cooled with

accelerated cooling (cooling by water jet). When column cool down using water jet, it experiences sudden shock resulting severe cracks than the room temperature cooling down.



**Figure 2.11** Fire test and loading frame (Adapted from Bikhiet *et al*., 2014)

### 2.6.1 Non-destructive Residual Strength Testing

Yaqub and Bailey (2016) have conducted an experimental study on the residual compressive strength of post heated concrete columns. They have used the non-destructive testing using ultrasonic pulse velocity test. This method can be effectively used for field applications. For the pulse velocity testing the transmitting and receiving transducers are placed on the opposite side of specimen. It is required to smoothen the surfaces of specimen, so usually petroleum jelly is used to apply on the surfaces. The distance between transducers divided by transit time to obtain the pulse velocity through the concrete. To analysis the damage in concrete, the structure can be divided into different zones such as inner zone, intermediate zone and outer zones. The inner zone has a temperature not more then 100°C and it has a damage factor of 1. The temperature for intermediate zone is in between 200-300°C and the damage factor is 0.85. And the most fire exposed zone, the outer region has a temperature of 300-500 °C with a damage factor of 0.4. when the temperature at outer surface exceeds 500 °C the damage factor considered as zero.
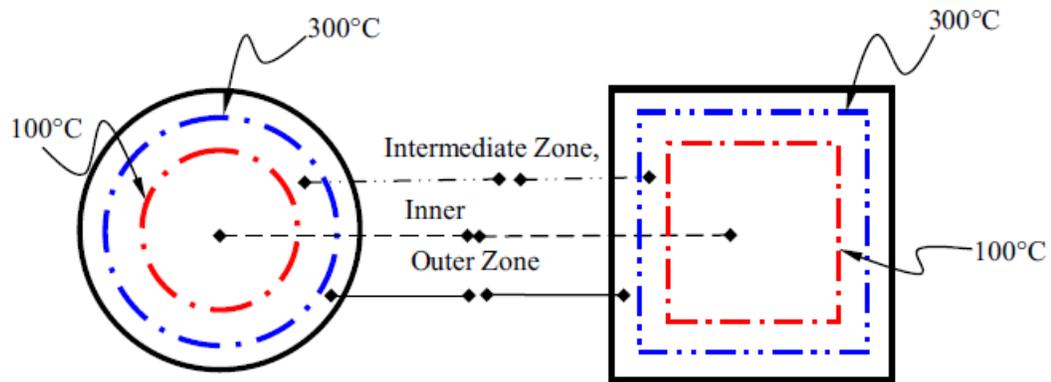
**Figure 2.12** Fire damaged zoning for circular and square columns

(Adapted from Yaqub and Bailey, 2016)

The testes were conducted on 35 reinforced concrete columns which includes square and circular columns along with three cubes for each column. The columns and cubes are heated to 500 °C at a rate of 150°C per hour. The pulse velocity measurement was conducted before, after and during the heating stage. After 7th day of testing, the concrete cubes tested under compression to obtain residual strength. There is no direct relationship between pulse velocity and residual compressive strength of heat damaged concrete columns. Hence, the research investigated to find a relationship between the ultrasonic velocity and residual compressive strength of heat damaged concrete cubes. The data from the experiments were used to develop a relationship between pulse velocity Vs temperature and residual strength Vs temperature. And finally, they also prepared a relationship between residual strength and pulse velocity. This relationship depends upon various factors such as concrete mix, types of aggregates, aggregate-cement ratio, water-cement ratio, age of concrete, size and grading of aggregate, curing conditions, air content, moisture content and density.

A non-destructive testing of fire-exposed columns was also conducted by Hibner (2017). The pulse travel time and pulse velocity are presented as a function of spacing which show a variation between prior to fire exposure and after fire damage. The variation in pulse time and velocity in fire exposed concrete column is due to the presence of cracks and shrinkages occurred in the specimen.

## 2.7 Finite Element Analysis (Numerical Analysis)

The experiments of fire testing are a time taking and expensive procedure. To overcome this finite element analysis can be used. There are several analytical studies are conducted to characterize fire resistance of reinforced concrete structures. However, only limited studies explain about residual strength of reinforced concrete columns after fire exposure. These analyses can be effectively used to predict the experimental behaviors of actual specimens and provides a deeper insight into different physical phenomena. Different software's are used for FEA, for e.g.: FORTRAN solver, ANSYS, ABAQUS, Microsoft Excel, MATLAB etc. The heat exposed RC structures involve hydro-thermo-mechanical problems. The heat transfer occurs mainly by conduction, convection and radiation. The temperature change across the cross section is analysis and along the length of the temperature is assumed to be constant. Usually the thermal analysis has carried out by neglecting the presence of steel rebars. From the thermal analysis we can determine the temperature at any point of the cross section and for any fire duration. (Bamonte and Monte, 2015)
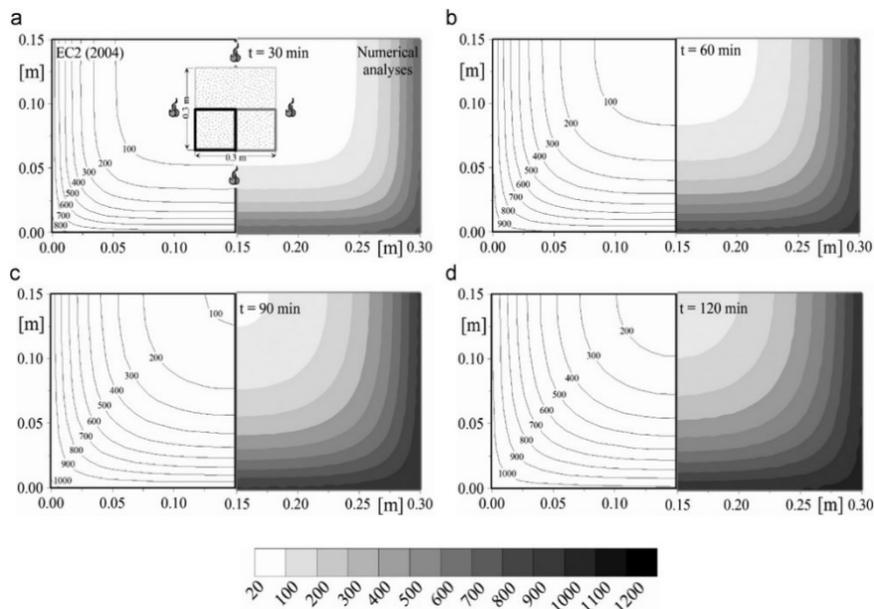


**Figure. 2.13** Square column: comparison between the thermal fields evaluated numerically (right) and given by EC2, at different fire durations: 30 (a), 60 (b), 90 (c) and 120 min (d). (Adapted from Bamonte and Monte, 2015)

The main step in thermal analysis is to determine the temperature propagation inside the cross-section. Material properties such as thermal conductivity and specific heat can be acquired from EC2.
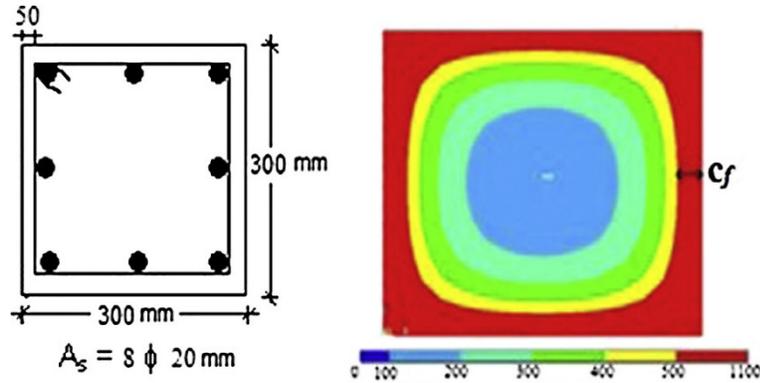


**Figure 2.14** Cross section of column and its thermal contour

(Adapted from Balaji *et al.,* 2016)

## 2.7.1 Mechanical Analysis

For mechanical analysis some assumptions are considered (a) plane sections remain plane (b) negligible shear deformation and spalling of the section (c) steel-concrete bond is perfect always(d) concrete in tension is neglected. (Bamonte and Monte, 2015). A numerical analysis is conducted in the research done by Adam *et al*. (2007) of RC columns strengthened with angles and strips. They have used ANSYS 2005 general purpose FE program. Only top1/8th of the specimen was used for modelling because of the symmetry of load and geometry.
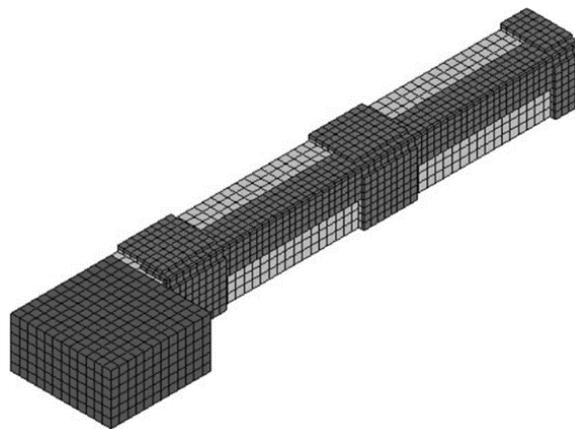


**Figure 2.15** Finite element model (Adapted from Adam *et al*, 2007)

The numerical analysis helps to develop some practical recommendations for the design of RC columns. The numerical analysis revealed that, increase in friction coefficient has caused significant increase in failure load. It is recommended to take extra care when setting mortar, avoid the formation of air bubbles or using expanding mortars. The numerical model also helped us to analyse the influence of mechanisms such as confinement pressure by strengthening and load transmission to the steel angles by friction, on the behaviour of RC columns.

Li *et al.* (2018) have performed finite element analysis on steel mesh reinforced resin concrete columns. They used ABAQUS software to develop concrete damaged plasticity model. Using C3D8R the normal concrete and resin concrete were modelled. The reinforcing bars and steel mesh were modelled using T3D2 element. The simulation results of the load-deflection relationship follow the same results obtained from experiments. Figure 2.9 shows the complete meshed finite element model.
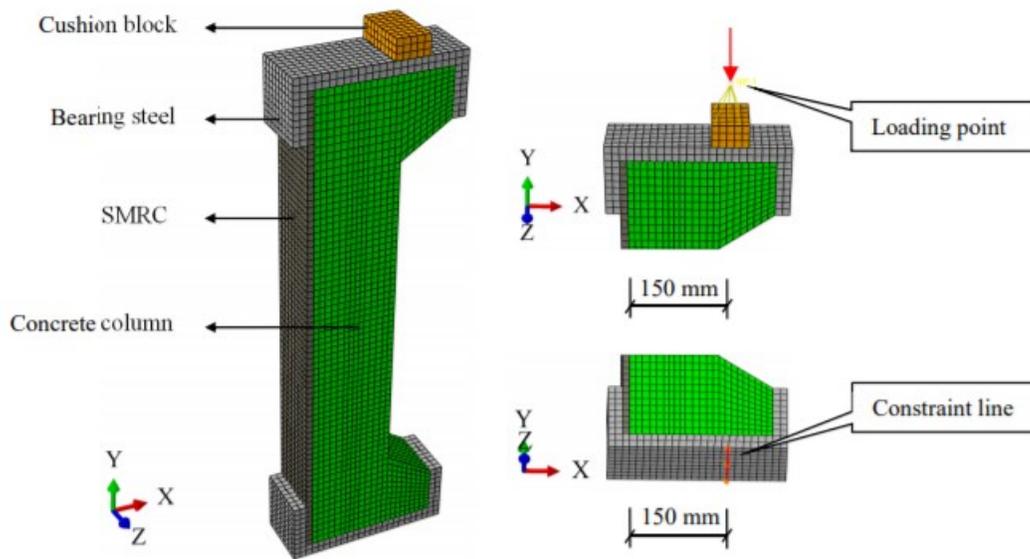


**Figure 2.16.** Finite element model and boundary condition (Adapted from Li *et al,* 2018)

In the experimental study the residual strength of blast damaged RC columns, Bao and Li (2010), had developed FEM program to analyse geometric modelling and impact analysis. The modelling

includes blast loading, structural geometry, material models, load applications and analysis procedures.
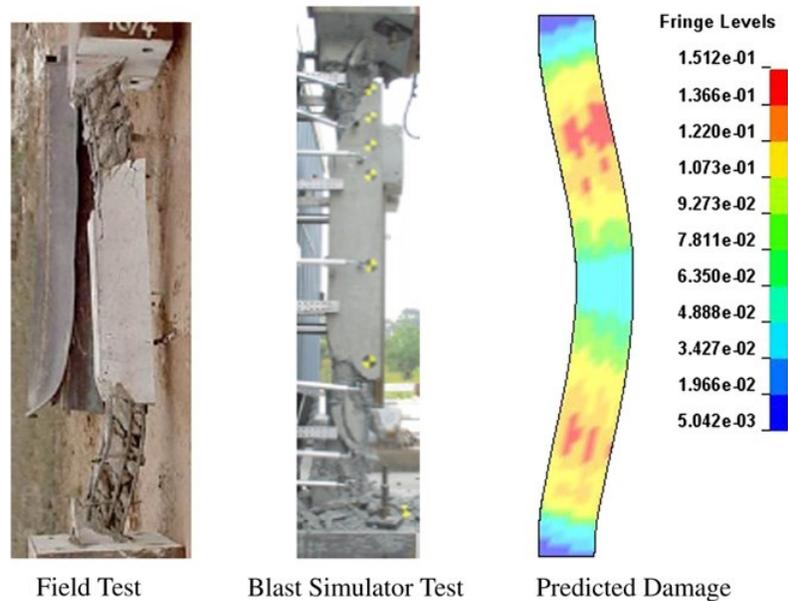


**Figure 2.17.** Comparison of numerical and experimental response of reinforced concrete columns (Adapted from Bao and Li, 2010)

The results from numerical analysis show that axial load ratio, longitudinal reinforcement ratio, transverse reinforcement ratio and column aspect ratio have influence on residual strength of blast damaged RC columns. The residual axial capacity ratio is smaller in case of larger axial loads at the same mid height displacement ratio. The longitudinal and transverse reinforcement are directly related to ratio of residual axial capacity. At high transverse reinforcement ratio, the residual capacity ratio increases with the reduction in aspect ratio. (Bao and Li, 2010).

## 2.8 Enhancing Structural Performance

Concrete structures are designed to have a safe life span of 50 years. Some structures remain more than its service span, but some destroys before the service time based upon different factors such as poor construction, poor design, inadequate material selection, severe environment than anticipated or a combination of these factors. The disadvantages of concrete such as brittleness, low tensile strength can be overcome using fiber reinforced concrete. Fiber reinforced concrete is defined as a concrete incorporating relatively short, discrete, discontinuous fibers. The main

purpose of adding fibers is to control the cracking of the FRC. Most commonly used fibers are steel, organic polymer, glass, carbon, asbestos and cellulose. All these fibers have different geometry, properties, effectiveness and cost. In a composite matrix the mechanical properties depend upon fiber, cementitious material as well as on the bonding between them. In FRC the main role of fiber is to bridge across the cracks.

Fiber reinforced polymers have been started using in the construction industry from the past two decades. Studies show that FRPs can be used to strengthen the concrete structures. The most commonly used technique is wrapping fiber-reinforced polymer around the concrete column that enhance the load bearing capacity. FRPs have some significant properties like high strength to weight ratio, stiffness to weight ratio, large deformation capacity, corrosion resistant, minimal change in the geometry and speed of application. However, at elevated temperature mechanical and combustible properties are significantly reduced due to the properties of the matrix resin. As such, concerns regarding fire resistance have slowed FRP applications in buildings.
Two types FRPs are commonly used; carbon fiber reinforced polymer and glass fiber reinforced polymer. The advantages of CFRPs are lightweight, high strength & stiffness and do not corrode. At the same time GFRPs have the properties such as non-reactive to chlorides, lightweight, lower shipping cost and easier construction practice.

## 2.8.1 Ambient Temperature Strengthening

Concrete structures are expected to perform well throughout its service life. The corrosion of reinforcing steel is one of the major problems facing by civil engineers. One American estimate shows that there is $150 billion worth of corrosion damage is occurring on highway bridges. In UK, the corrosion induced damage is a total of £616.5 millions. Poor supervision and poor-quality control lead to the production of poor-quality concrete and low concrete cover which results in the corrosion of steel.

Different strengthening techniques are adopted for RC columns. The most commonly used methods are concrete jacketing, composite-based strengthening systems (FRP) and the use of steel jacketing. Steel jacketing is the most commonly used method. Usage of steel angles and stirrups

are mainly used in Europe. A paper has studied the effect of metal strips and angles on the behaviour or RC columns. Steel strips at the column ends exert confinement pressure on concrete and thus the failure load can be increased. The cement mortar also has important role in load transmission between cement material and strengthening material. (Adam *et al.*, 2007)

The use fiber reinforced polymers such as carbon fiber reinforced polymers (CFRPs), glass fiber reinforced polymers (GFRPs) are normally used in concrete for strengthening. These materials can be used as an effective alternative to steel. GFRPs are good corrosion resistant, high tensile strength-weight ratio, nonmagnetic and non-conductive. The fibers can provide a considerable amount of post-cracking ductility. In steel reinforced concrete, the fibers not only reduce cracking, they increase the bond between steel bars and concrete.

A research was carried out to improve the seismic strength and cyclic load reversals of RC columns by Cho *et al.* (2018). Old RC column surface was grooved, and longitudinal and transverse reinforcements placed in the grooves. The surface finish was done by spraying mortar with high performance fiber mortar. In order to prevent the collapse of a structure under earthquake, the column in lower stories should have sufficient inelastic deformation capacity. The lateral deformation and load carrying capacity can be enhanced by transverse reinforcement which also provides confinement to the core, works as shear reinforcement and prevent longitudinal reinforcing bars from premature buckling. High performance fiber reinforced concrete (HPFRC) mortar enhances the brittleness of concrete in tension. Under uniaxial tension HPFRC exhibits pseudo strain hardening behaviour. They have used poly vinyl alcohol fiber and high strength poly ethylene fibers. And they obtained a result that overall load carrying capacity is improved by this method. It prevents shear failure, minimize the bending cracks, buckling of longitudinal bars and reduced the spalling of cover.

Cho *et al.* (2012) have also done a research on HPFRC mortar on RC columns. In this experiment they mostly concentrated on the plastic hinge region of RC columns which lack the lateral loading and deformation capacity.
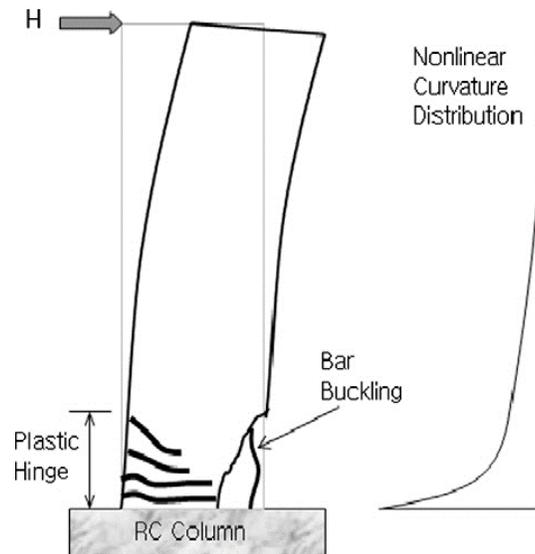
**Figurer 2.18** Plastic hinge region (Adapted from Cho *et al.*, 2012)

The main aim of this research is to develop seismic strengthened RC columns by applying HPFRC mortar on the plastic hinge region. The experimental research indicates that, the method of applying HPFRC on the plastic hinge regions reduced the concentrated local damage and the overall lateral performance has improved.

Some of the advanced materials used for concrete column strengthening are Polymer concrete, SIFCON, DUCON, UHPC (Roller *et al.*, 2013).

Polymer concrete: (Figure 2.17(a)) It can absorb strong shock waves. To absorb kinetic energy and convert it into another form of energy the material should have some properties such as capability of compaction and enough ductility. These properties are attained by two components; porosity which withstand compaction and achieved by adding porous organic fillers whereas, ductility is achieved with resin and flax fibers. Together with this hardness and fine-grained fillers like calcspar and quartz sand are used.

SIFCON (Slurry infiltrated fiber concrete): (Figure 2.17(b)) It is a type of High-performance fiber reinforced concrete (HPFRC) which contains short steel fiber segments. SIFCON consists of a bedding of steel fibers infiltrated with a slurry of cement, fly ash, and water. This tough material

absorbs large amount of energy. This steel fibers have a great influence on compressive strength than the conventional concrete. This SIFCON can be used as a strengthening material.

DUCON (Ductile concrete): (Figure 2.17(c)) It consists of micro 3D reinforcement and an optimized self comacting high strength cement. The micro-reinforcements results the spatial distribution of load capacity. This material has high compressive strength, high tensile strenght as well as high ductility. The mesh size has influence in the strenght. The material can be made into different strength by adjustin both slurry and reinforcemnt.

UHPC (Ultra high performance concrete): (Figure 2.17(d))  It is a mixture of high strength cement, aggregates and reactive micro components like quartz and silica dust. UHPC has properties of high compressive strength, fracture energy and stiffness. This results increase in load bearing capacity.



(a)                              (b)

(c)                              (d)
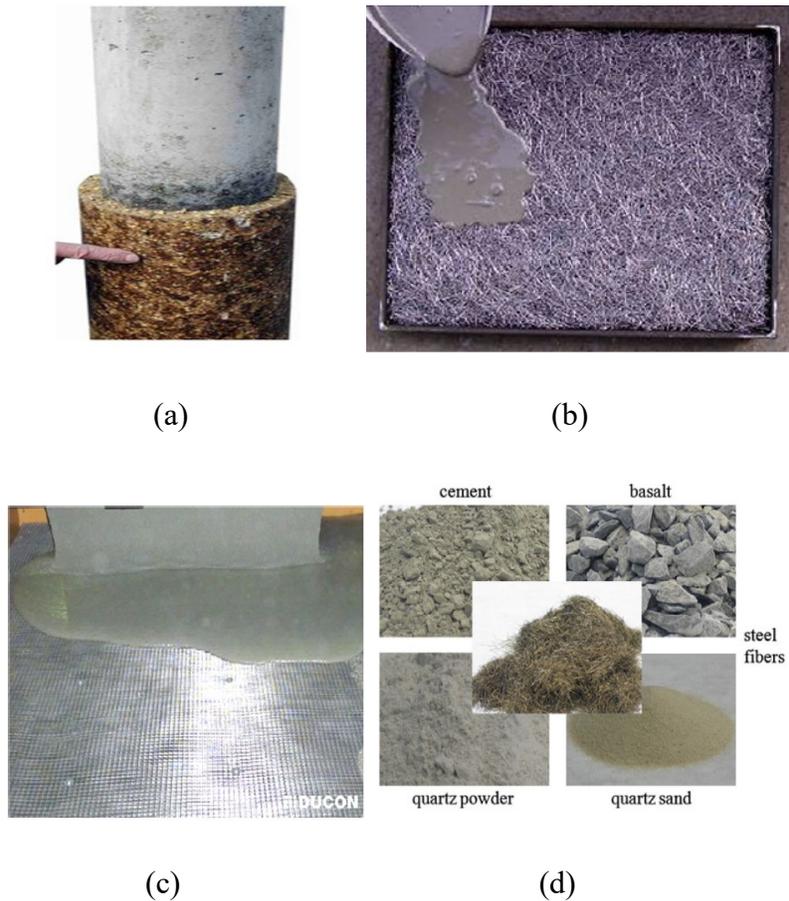
**Figure 2.19** Advanced strengthening materials (a) Polymer concrete; (b) SIFCON; (c) DUCON; (d) Single components of UPHC (Adapted from Roller *et al.*, 2013)

## 2.8.2 Elevated Temperature Strengthening

Structural fire safety is really important for the building and the fire resistance should be adequate for the structural integrity until the fire situation comes under control. An experimental study conducted by Green *et al*. (2006) suggest that the use of FRP is an effective method. The thermal insulation of FRP material helps to prevent the rapid loss of the structural effectiveness of FRP system during fire. The experimental results reveal that the temperature of insulated concrete and steel reinforcement remained less than 400°C during the fire exposure. Temperature less than 400°C is not significantly considered for concrete or reinforcing steel. The columns retained their unconfined strength during the fire exposure.   To improve the fire behaviour of reinforced concrete structures, the concrete can be strengthened with fiber reinforced polymers.
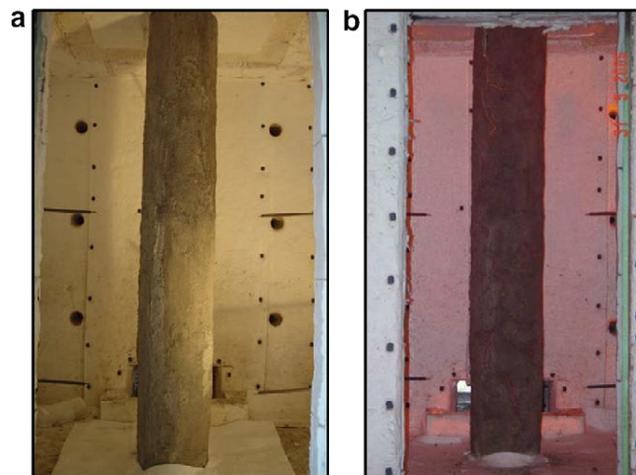


**Figure 2.20** Insulated FRP wrapped column (before and after fire exposure)

(Adapted from Green *et al*., 2006)

FRP material was believed to be less useful in fire because of the deterioration of the material at glass transition temperature which is about less than 100°C. But the FRP wrapped columns performed well in fire. The performance has much more improved when they used a supplemental fire insulation system applied over to the exterior of FRP wraps.

The experimental research performed by Rodrigues *et al.* (2010) explained the behaviour of fiber reinforced RC column exposed to fire. They used steel fibers and poly propylene fibers along with the steel reinforcement in the concrete matrix. The water content in a concrete change its state to

vapour at high temperature and it results high pressure in the concrete. The water vapor tries to escape from the concrete through the pores. If concrete resist the escape of vapour that leads to the internal pressure development and causes explosive spalling of concrete. The experiment result has proven that the concrete with poly propylene fiber and steel fibers did not show spalling after fire test.

Han *et al*. (2005) has also investigated the fire performance of concrete with poly propylene and lateral confinement. High performance concrete with poly propylene fibers and lateral confinement with metal lath, glass fiber and carbon fiber are used to prepare the specimens. The results show that concrete with poly propylene fibers and lateral confinement can prevent the spalling and have an efficient performance in fire. Also, among the lateral confinement materials the metal fabric is the best one. At 170°C the poly propylene fibers start melting and above 340°C it vaporizes. And thus, enough void spaces form, and it help the escape of water vapour.

## 2.9 Fire Protection Options

Fire resistance is the ability of a material to withstand fire. The fire resistance is defined as the time in hours during which a building element or an assembly maintains the ability to confine a fire, continuous to perform a given structural function, or both. Generally, 1 to 4 hours is the required fire resistance. Fire resistance of concrete is influenced by several factors such as type of concrete, dimensions, reinforcing steel, cover, structural restrains, eccentricity etc. A strengthening system used in the study was Sika Wrap Hex 103C unidirectional carbon FRP composite strengthening system. Sikacretes-213F was used as a fire protection system in one of the experiments. The studies showed that a fire insulating material used along with a CFRP material will have a significant impact on the fire characteristics of the concrete columns as it helps maintain the properties of CFRP beyond the glass transition temperature.

Green *et al*. (2006) had also used supplemental fire insulation system along with FRP wraps which showed much more fire resistance. This system consists of spray-applied cementitious mortar with specializes fillers and coatings. A clear idea of this insulation system is explained by Williams *et al*. (2006). This was developed by FyFe Co. LLC. This system consists of a layer of VG insulation and EI coating. VG is a spray-applied fire-resistant plaster which is applied to the exterior of FRP

wraps and EI is an intumescent epoxy surface-hardening coating which is applied to the outside surface of VG. During exposure, the EI coating activates with in 5 minutes (235°C). Another material used for insulation is a Portland cement-based mortar incorporating light weight fillers which was applied on to the exterior surface of FRP. Both of this insulation methods work extremely well during fire exposure.

A recent study on alkali activated lightweight mortars have been done by Carabba *et al.* (2019) for a passive fire protection option. The results indicate that all the specimens have high weight stability at high temperature. Geopolymers and alkali activated materials (AAM) are under investigations and to be used as an alternative to the conventional cement-based materials. The AAM are highly resistant to high temperature. At high temperature it under goes dehydration of weakly bound water in the gel and maintains the strength. The AAM is produced by the room temperature activation of coal fly ash.

# Chapter 3   Research Methodology

## 3.1 General

This chapter outlines the methodology used in this research study. The methodology includes the pre-calculations, designing and finalising reinforced concrete column dimensions according to the facilities available and code requirements, and ultimately experimentally testing the fabricated specimens at both ambient and elevated temperatures. The length of time from the beginning of fire exposure to the failure is the fire-resistance rating of a column. Design and construction of structures should be based on prescriptive or performance-based design approaches. Achieving code specified fire-resistance rating is the goal in prescriptive based design. For a reinforced concrete column, minimum dimensions of concrete cover, column dimensions, tie spacing, etc. are specified. As the column is the primary structural elements which transfers the building loads to the foundations, it's design must be safe and serviceable. The overall strength of reinforced concrete column affected by numerous factors, e.g. load, length, cross section, eccentricity, end conditions and concrete and steel strength.

There was only a limited number of experimental studies that have been carried out to evaluate the fire behaviour of reinforced concrete columns as well as post-fire behaviour. The main aim of the study presented in this thesis is to study the residual compressive strength of reinforced concrete columns which are subjected to a standard fire scenario. In a real fire situation, a certain amount of load is acting on reinforced concrete columns during the fire exposure. Even after fire hazard, the concrete columns retain some of its strength which needs to be evaluated to restore its serviceability or to define a retrofitting procedure. Fire and its effects of structural elements are complex, and thus the degradation of the material properties of concrete and reinforcing steel change as a result of elevated temperatures of fire in a non-linear manner. Accordingly, conducting experimental testing on reasonably sized columns to study as many as possible of factors affecting the behaviour and residual compressive strength of fire-damaged reinforced concrete columns, i.e., fire duration and load ratio, is very crucial.

## 3.2 Experimental Program

This thesis presents the results of a unique experimental study conducted on ten reinforced concrete columns: two were tested at ambient temperature until failure as control specimens; and eight columns were exposed to standard fire. According to the outcomes of several studies available in the literature, there is an excellent correlation between full-scale column models and smaller scale test specimens in terms of failure modes and cracking pattern (Bikhiet *et al.*, 2014). As a result, smaller scale column test specimens can be used as long as actual end restraining, and thermal boundary conditions are maintained. Accordingly, the column specimens experimentally tested in this study were cast with cross-sectional dimensions of 200 x 200 mm and 1500mm long, to fit in a vertical alignment inside the fire testing furnace.

The experiments of this study were performed at Lakehead University Fire Testing and Research Laboratory (LUFTRL), Thunder Bay campus. All column test specimens had identical geometry, reinforcement details, and concrete mix. Descriptions of test specimens and experimental program are detailed in the following sections.

## 3.2.1 Test Specimens

All column specimens were designed as per CAN/CSA A23.3-14 standards and the details of the column design calculations are explained in Appendix A. Minimum steel reinforcement of 400 MPa yield strength was provided, consisting of four 10-mm diameter longitudinal rebar and 10-mm diameter stirrups placed at 200 mm center to center spacings. The reinforcement was designed with a concrete cover of 30 mm which is a typical cover thickness used in practice. The stirrups were bent at $90^0$ to the concrete core. The concrete mix used in casting all concrete column specimens had characteristic compressive strength of 36.4 MPa verified by testing six concrete cylinders after been cured for 28 days. Since concrete gain strength as aging, the compressive strength of concrete increased to 50 MPa on the test day which is about 6 months after casting. The maximum axial compressive design capacity of the 200 x 200 x 1500mm long columns was determined as 940 kN. The concrete column specimens were stored indoor at dry conditions for about six months to guarantee diminish of moisture contents, and thus avoid spalling issues when tested at elevated temperatures. The elevation, cross sections, and instrumentation arrangements for the fabricated columns are shown in Figure 3.1.
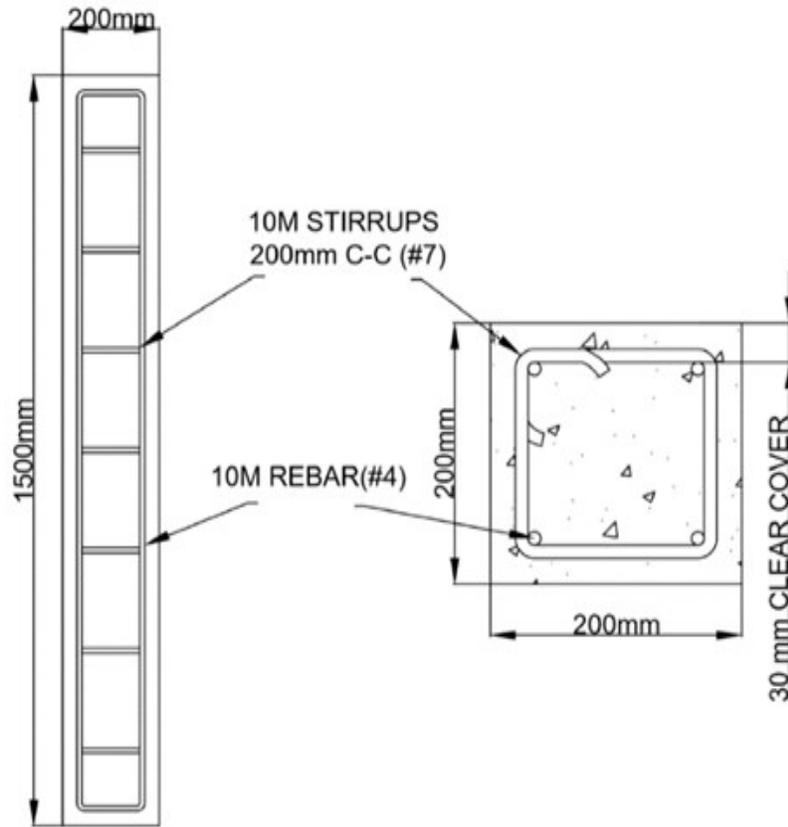
**Figure 3.1.** Dimensions and reinforcement details of reinforced concrete columns

### 3.2.2 Formworks for Casting Specimen

Each column was cast horizontally in the formworks made of plywood and dimension lumber. The design of formworks is detailed in Appendix C. All plywood pieces were measured and cut accurately using ta table saw and fastened with construction screws. The cross ties fixed after placing the reinforcement cages in its places inside the formworks. The formworks were made sufficiently rigid and strong enough to withstand handling without losing its dimensional integrity and were placed on a flat surface before pouring concrete. Sealant was applied along the plywood joint lines in the formworks to make them watertight. Before placing concrete, a construction mineral oil was spread on all internal surfaces of the formworks to ease its removal from casted concrete specimens afterwards. The reinforcement cages were aligned horizontally in such a way that it were hanged inside the formworks maintaining a clear cover of 30 mm from all four sides.

Concrete was poured in layers while being vibrated using portable vibrator. Figure 3.2 shows a sample of a formwork that accommodated three column specimens and the reinforcement cages just before placing concrete.
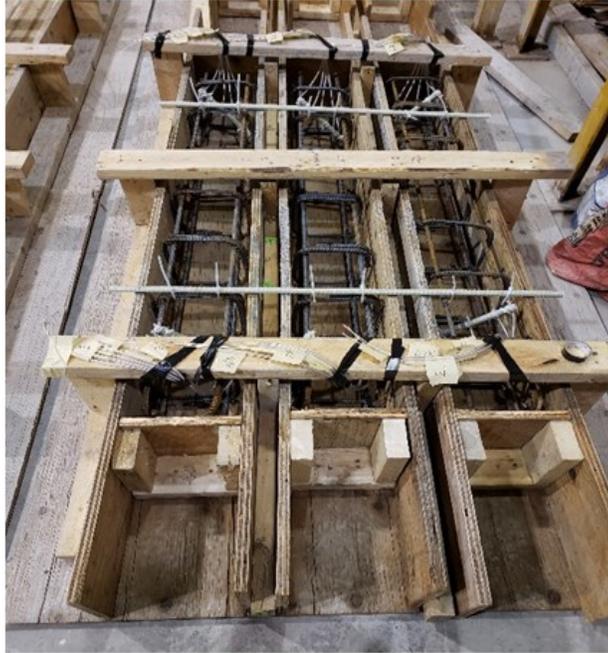


**Figure 3.2.** Formwork and reinforcement of cast columns

### 3.2.3 Instrumentation

Instrumentation for the columns included thermocouples, strain gauges, and linear variable displacement transducers. The two columns tested at ambient temperatures were instrumented with two strain gauges each to capture the compressive strains, whereas the other eight columns tested at elevated temperatures were instrumented with ten thermocouples each, at two different elevations, 200-mm distance from each end, in a diagonal manner as shown in Figure 3.3. High temperature glass-fiber insulated K-type thermocouples were utilized to measure the temperatures at various depths inside the concrete. A short FRP bar segment was attached diagonally to two of the steel reinforcing bars to keep the thermocouples in place. Also, each thermocouple was inserted inside a protective PVC tube to avoid concrete environment incursions. The thermocouples were arranged in such a way that it can measure the temperature attained at external surface of the tie, rebar, and center of each cross section. Since the FRP bar segment used to secure the

thermocouples in place is non-conductive, the chances of unnecessary heat conduction and noise in thermocouple readings were avoided.
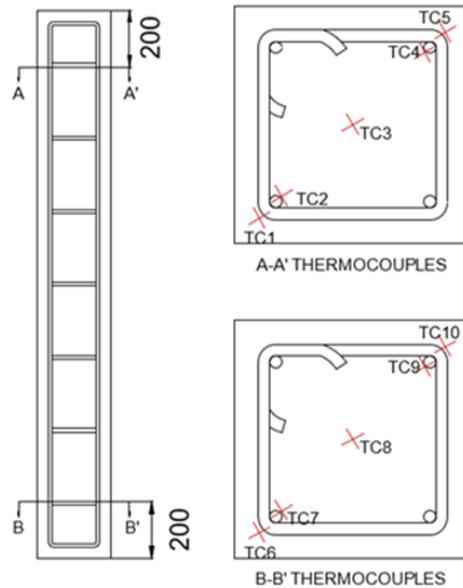


**Figure 3.3.** Thermocouples layout at two different elevations of a general test specimen

Each K-type thermocouple consists of a pair of wires which are of different metals and connected by a welded bead. These two wires are individually insulated and the external layer of insulation surrounds both wires. It was necessary to remove both layers of insulation of a maximum of ½ inch, then hold the thermocouple wires securely near where the insulation removed and make the two wires in contact at the edge. Afterwards, the wires were welded together using fine-gauge wire welding machine to form a bead. This welded bead must come in direct contact with the material to read the temperature at any given point inside the concrete column specimen. It was important to spend enough time and effort to check and prepare thermocouple as much as possible. Figure 3.4 shows the fine-gauge wire welding machine and a typical thermocouple bead prepared using the welder.

(a)                                                    (b)

**Figure 3.4.** Thermocouples preparation, (a) Fine-gauge wire welding machine; (b) a typical thermocouple bead

Figure 3.5 depicts an example of the installation of the thermocouples in a general column's formwork. Each thermocouple wires attached to an FRP rebar using zip ties and then attached to the reinforcement cage as shown in Figure 3.5.
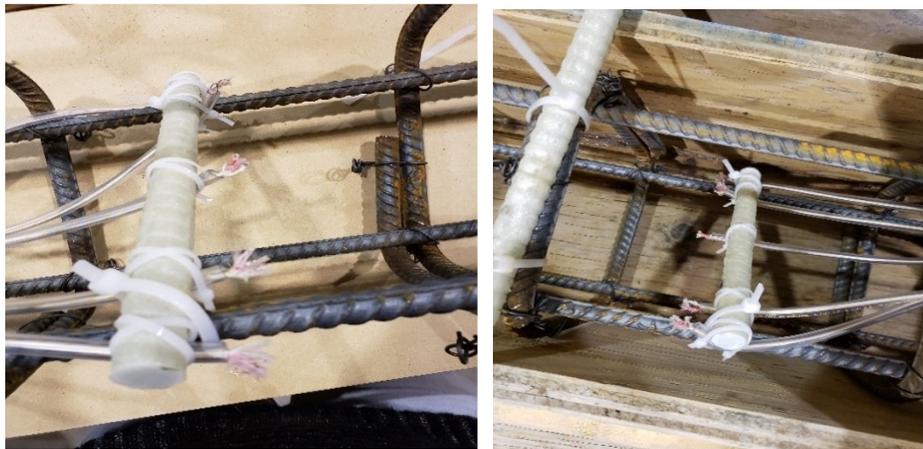


**Figure 3.5.** Thermocouple installation in a general column's formwork

## 3.3 Ambient Temperature Tests

Normal temperature testing was conducted at Lakehead University's Civil Engineering Structures Laboratory. The columns were tested under compression loading using 500-kip large Universal Testing Machine (UTM) until failure. The axial deformations were measured at the top of the column using a built-in draw-wire displacement transducer as part of the universal testing machine (UTM) utilized to load the two column specimens tested at ambient temperature until failure. Each test specimen was instrumented with two strain gauges that were mounted on the longitudinal rebar at a height of one third from bottom to capture the compressive strains during loading. Each test was duplicated to verify the experimental results. The data obtained from the ambient temperature tests were used to plot the stress-strain curves and axial deformations curves for both specimens. The failure pattern was observed, and the failure load and maximum axial deformation were noted.



**Figure 3.6.** Ambient temperature test setup

To provide a flat surface at both ends of the column specimens and to ensure concentric load application during testing, capping of both ends of each test specimen using Gray Iron 9000 (sulfur cement) capping compound, which consists of ultra-thin and low-odour flakes produced from sulfur and mineral filler, was necessary. This capping material melts slowly until the optimum pouring temperature of about 260°F - 290°F is reached. To melt the capping compound material, a thermostatically controlled electric melting pot was used. The compound material would not affect the strength nor the bond properties of the capped specimens. Overheating (above 300°F) may cause thickening of the capping compound material; however, the material can be still used if overheated to a temperature but not exceeded 320°F. Each column specimen was sprayed with

a thin layer of mineral oil on the surfaces that were in contact with the capping compound. Pour the proper amount of mixture to the capping rig and place the specimen immediately in position for the thickness of the cap desired. Figure 3.7 shows the custom fixture used for our specimen which was prepared in the lab along with a prepared capped end.

The main features of the capping compound are:

- ASTM C617 compliant;
- Compound can be re-melted with no loss of properties;
- Melt quickly at 110 - 115°C and lower optimum pour temperature.



(a)    (b)
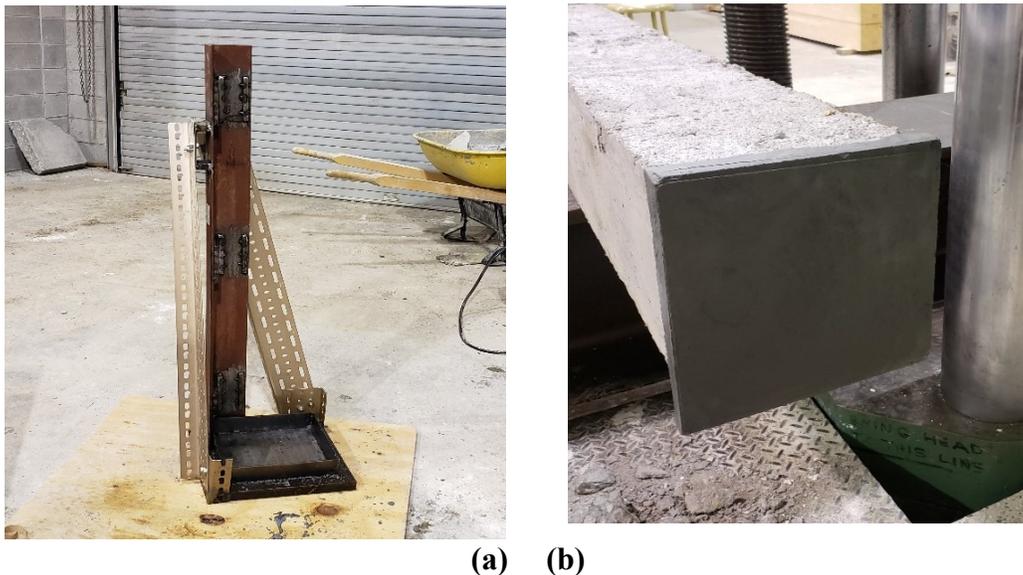
**Figure 3.7.** RC column end compound capping process, (a) capping rig; (b) prepared column end capping.

## 3.4 Elevated Temperatures Tests

Fire tests were carried out at the large-size fire testing furnace accommodated at Lakehead University's Fire Testing and Research Laboratory (LUFTRL), Figure 3.8. The state-of-the-art custom-designed furnace is equipped with two natural-gas fed burners that can raise the furnace temperature up to 1300°C. The unique design of the furnace chamber permits uniform temperature distribution across the furnace. The furnace has movable front large door that enables the insertion

of large test specimens, Figure 3.9. In addition, each of the furnace's roof and floor is provided with three 400-mm square vents to facilitate the insertion of long specimens, such as columns, as well as to apply transverse load and accommodate end supporting restraints during fire tests.



**Figure 3.8.** Lakehead University Fire Testing and Research Laboratory (LUFTRL)



**Figure 3.9.** Large custom-designed fire testing furnace

Full control of the environment temperature inside the furnace was facilitated via advanced control panel that is equipped with Human-Machine Interface (HMI) touch screen, Figure 3.10. The furnace inside temperatures were measured using three metal shielded thermocouples that are built in the furnace by which thermal feedback interpreted by the furnace control panel allows the adjustment of the gas flow to the burners to maintain the desired standard fire profile.



**Figure 3.10.** Human-Machine Interface (HMI) touch screen
of the furnace's control panel

### 3.4.1 Elevated Temperatures Tests Setup

Each column was placed inside the furnace under concentric axial loading with both ends assumed pinned. Two columns were tested at a time, with all four faces of each column were subjected to fire. The fire testing furnace has a clear internal height of 1600 mm, and thus all four faces along the entire column length was exposed to fire. Two small viewports are provided on the furnace's front door which enables monitoring specimens during tests, Figure 3.11. The pre-determined time-temperature profile selected using the HMI screen of the furnace control panel was used to control the furnace temperatures.

**Figure 3.11** Viewport during a fire test

A total of twenty thermocouple wires were inserted into the furnace through the top and bottom vents and plugged to the respective. Each wire was plugged in and attached to the Data Acquisition system. Each column was placed in position under concentric axial load with both ends pinned. Top and bottom of all columns were protected with a ceramic fiber blanket of 150-mm width. A load ratio of 20% and 40% of the maximum design capacity of the columns at ambient temperature was maintained throughout the fire test that lasted either one or two hours. The load was applied using two hydraulic jacks that were mounted to the steel strong loading structure. A total of eight fire tests were conducted in which four are the duplication of the first sets of four tests. The results presented herein are the average of the two sets of experiments. A layout of the fire test setup is shown in Figures 3.12 and 3.13.
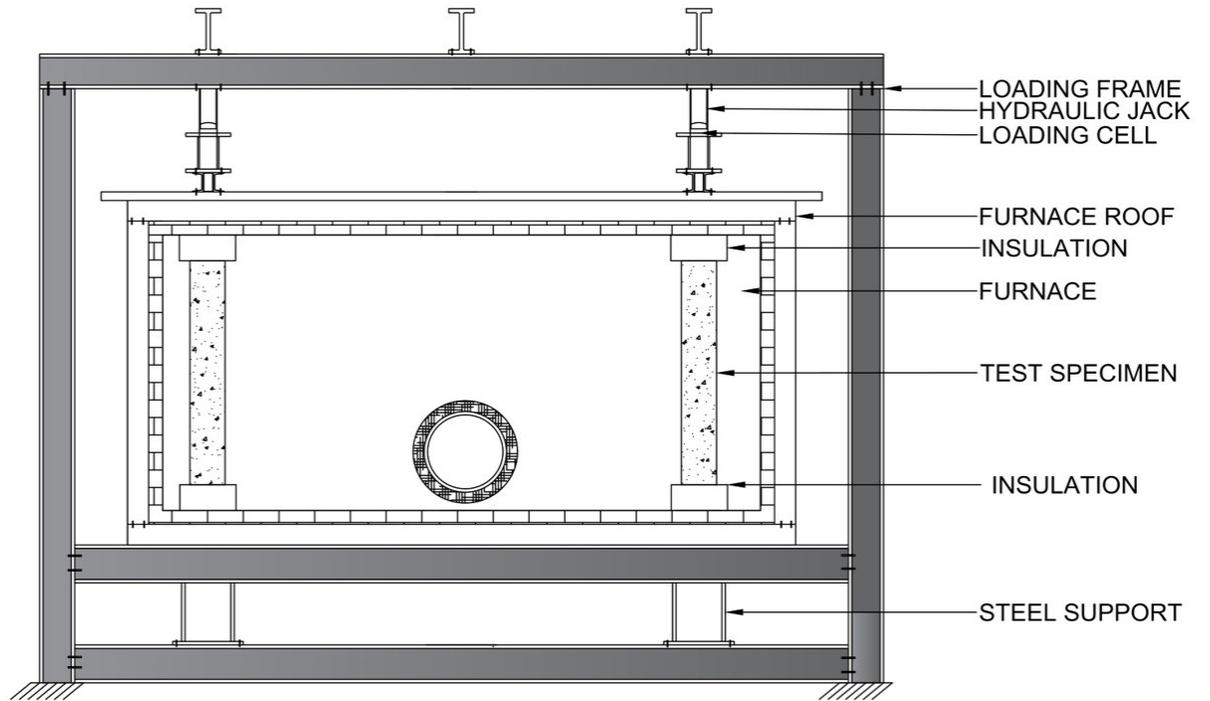
**Figure 3.12.** Schematic of a general fire test setup

Labels in figure:
LOADING FRAME
HYDRAULIC JACK
LOADING CELL
FURNACE ROOF
INSULATION
FURNACE
TEST SPECIMEN
INSULATION
STEEL SUPPORT



**Figure 3.13.** LUFTRL's fire testing furnace with two columns placed inside

Figure 3.13 shows the image of two columns setup inside the furnace. The hydraulic jacks attached to the loading frame applied the load 30 minutes before the start of the fire test, as per CAN/ULC-S101 fire endurance testing standard and was maintained throughout the fire test duration. In order to establish a realistic fire condition, the columns were loaded with two different load intensity ratios (20% and 40%) of the column's ultimate design capacity during fire tests for two different fire durations (1 hour and 2 hours). Accordingly, there were four different fire tests and duplication of all tests was conducted.

### 3.4.2 Elevated Temperatures Data Collection

The twenty thermocouples implemented inside each test specimens and attached to the data acquisition system were utilized to monitor the thermal profile of the column test specimens during the heating and cooling phases, Figure 3.14.
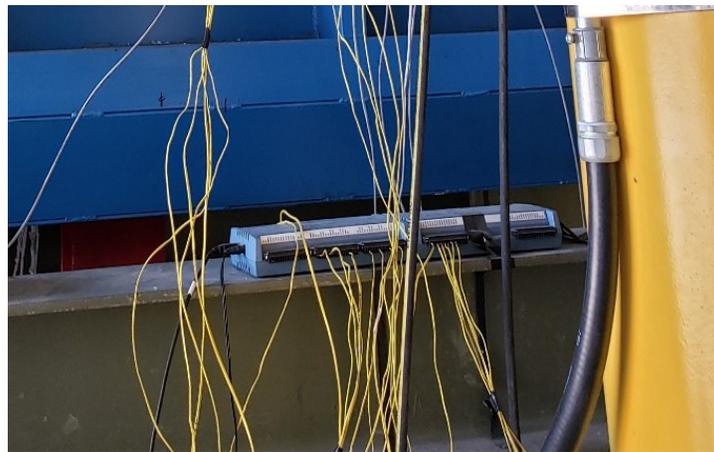


**Figure 3.14.** Thermocouple wires attached to the data acquisition system

### 3.5    Residual Strength Testing

After completion of the fire tests, the furnace was allowed to cool down and then columns were visually inspected to check if any spalling or cracks might have developed. In a subsequent phase, residual compressive strength tests were conducted for all post-fire exposed columns. During residual strength tests, the load was applied on the columns following a displacement-control protocol at the rate of 0.5 mm/minute until failure. Axial deformations and failure pattern were monitored and documented throughout tests. Figure 3.15 shows the setup of a general fire-damaged column undergoing residual strength testing using the large UTM.

**Figure 3.15** Axial compression test setup

# Chapter 4   Ambient and Elevated Temperatures Tests Results

This chapter discusses the results of the tests that performed on the reinforced concrete columns both at ambient and elevated temperatures. This includes the structural response and thermal response of the RC columns during testing. Compressive strength of RC columns from ambient temperature testing work as a benchmark for the study. The analysis of this section is necessary in order to determine the acceptability of the experimental results and the subsequent column behavior study.

## 4.1     Ambient Temperature Tests Results

The two RC columns tested at ambient temperature are considered as control specimens to obtain an average of the ultimate compressive failure load of the columns. With the help of axial deformations and strain measurements, load-deformation curves and stress-strain curves for RC columns were plotted. Figure 4.1 shows the RC columns after failure under concentric axial compression loading.



**Figure 4.1** Crushing failure of one of the two columns tested at ambient temperature

The structural response of the RC columns can be determined by plotting measured axial deformations with respect to the concentric load applied. Figure 4.2 shows the axial deformation-load curve in which Column (A) measured a maximum axial deformation of 3.82 mm at a failure load of 1933.6 kN, whereas Column (B) measured a maximum axial deformation of 4.4 mm at a failure load of 1825 kN. Both columns tested at ambient temperature has shown very similar trend lines for load-deformation relationships.
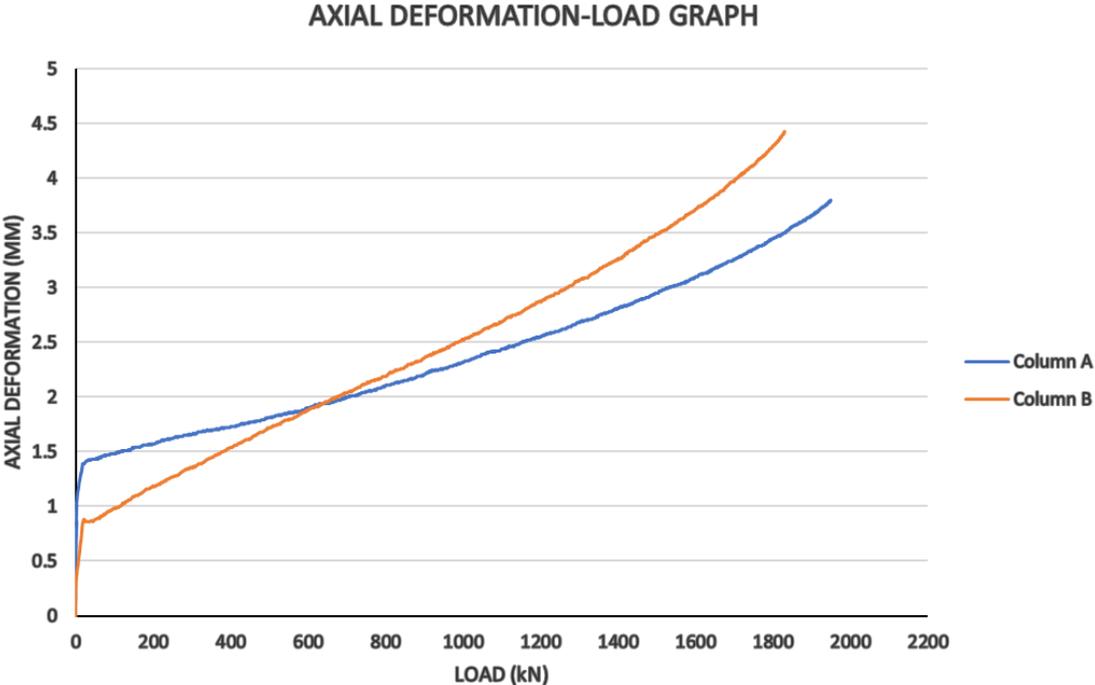


**Figure 4.2.** Ambient temperature tests axial deformation-load relationships

The strains developed in the longitudinal reinforcement of the columns were monitored during the ambient temperature tests. The strain gauges were installed at one-third height of the column from its bottom end. The developed stress-strain relationships using the measurements of the two columns are presented in Figure 4.3. The stress in the column obtained from the corresponding load value and cross-sectional area. The stress-strain curve followed a straight line during the elastic region. The stress is proportional to strain within the elastic region. Beyond the elastic region, it showed plastic behavior and the curve reached maximum compressive strength. After reaching maximum stress, the curve started to descend.
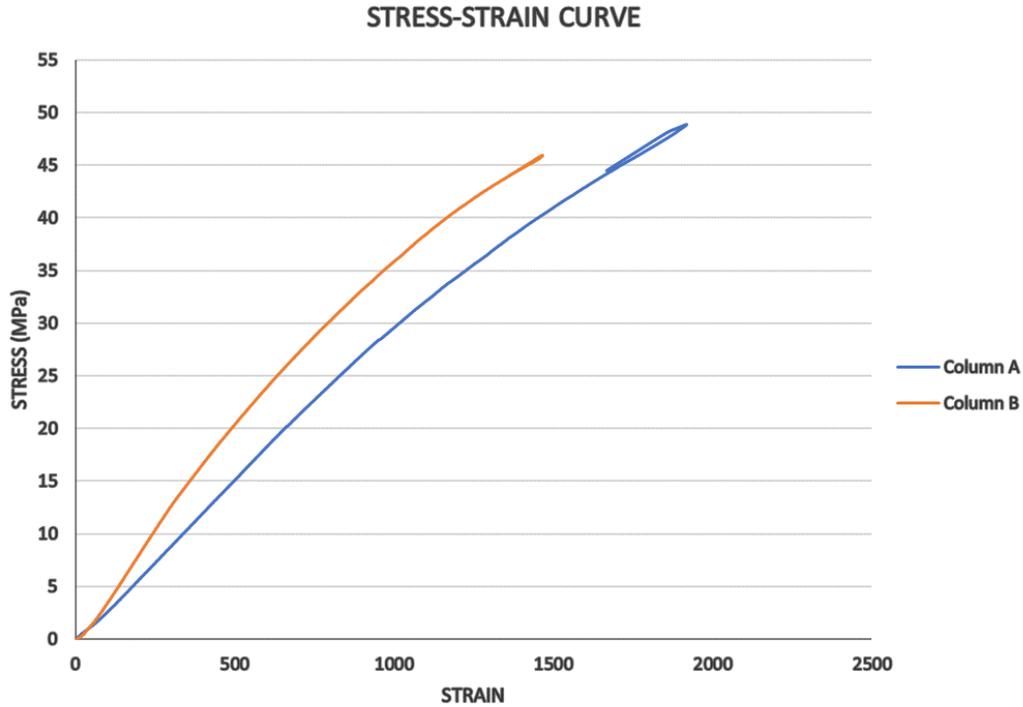
**Figure 4.3.** Ambient temperature tests stress-strain relationships

The experimental results obtained from the two ambient temperature tests are summarized below in Table 4.1.

**Table 4.1.** Ambient temperature tests results

| Column designation | Concrete strength (MPa) | Design capacity (CSA- A23.3) | Ultimate failure load (kN) |
|---|---|---|---|
| C-1 | 50 | 940.0 | 1933.6 |
| C-2 | 50 | 940.0 | 1825.1 |

The design capacity of column obtained as 940 kN using the characteristic compressive strength of concrete (50 MPa) on the test date. The average value of the ultimate compressive load capacity of the two columns is calculated at 1879.3 kN, and both columns failed due to crushing. There is a significant difference between the column's maximum design capacity and its ultimate failure load due to the strength reduction factors accounted in the design, as well as the variation between the actual compressive strength of concrete and the strength design value.

## 4.2    Elevated Temperatures Tests Results

Four of the eight column test specimens tested at elevated temperatures that followed a standard fire profile were subjected to monotonic concentric compressive load that was applied 30 minutes before the start of the fire test, as per CAN/ULC-S101 standard, and was equal to about 20% of the column's maximum design capacity. The applied load (180 kN) was maintained throughout the entire duration of the fire tests. The other four columns were subjected to loads of higher level that was equal to 40% of the column's maximum design capacity (360 kN). Two specimens of each of the column groups were exposed to one-hour fire test duration, whereas the other two specimens were exposed to two-hour fire test duration. The fire test duration considerably affected the peak temperature attained inside the concrete section as well as the residual compressive strength of the columns. Fire tests matrix is shown below in Table 4.2.

Table 4.2. Fire tests matrix

| Column designation | Fire duration (hours) | Applied load during fire (kN) |
|---|---|---|
| C1(A)-20% | | 180.0 |
| C1(B)-20% | | 180.0 |
| C2(A)-40% | 1.0 | 360.0 |
| C2(B)-40% | | 360.0 |
| C3(A)-20% | | 180.0 |
| C3(B)-20% | | 180.0 |
| C4(A)-40% | 2.0 | 360.0 |
| C4(B)-40% | | 360.0 |

### 4.2.1 Thermal Response

The thermal propagation inside the concrete sections was captured by ten K-type thermocouples installed inside each column test specimen. The thermocouples were installed at two different sections, each at 200-mm distance from each end of the column. Figures 4.4 and 4.5 show the standard fire curve, furnace average temperatures, and the temperatures measured by the thermocouples at the top and bottom sections, respectively, for a sample column exposed to 1-hour fire duration test. Similarly, Figures 4.6 and 4.7 show the temperatures measured by the thermocouples at the top and bottom sections, respectively, for a sample column exposed to 2-hour fire duration test.
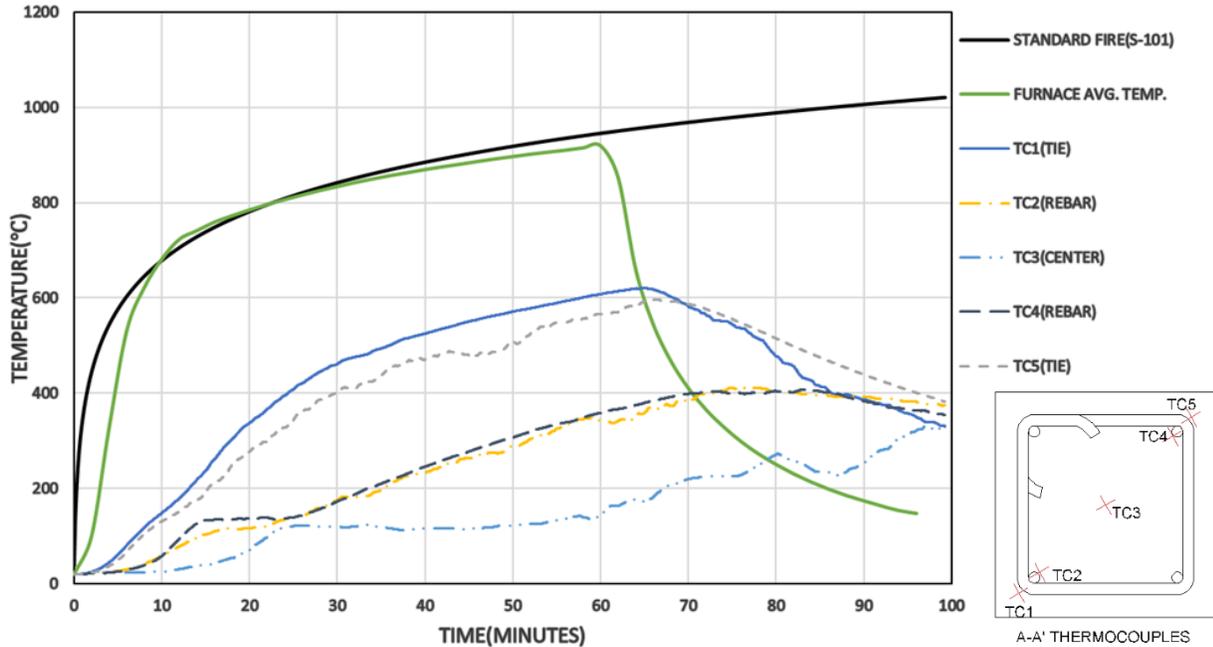


**Figure 4.4.** Time-temperature curves for a 1-hour fire duration test (developed using thermocouples at section A-A')
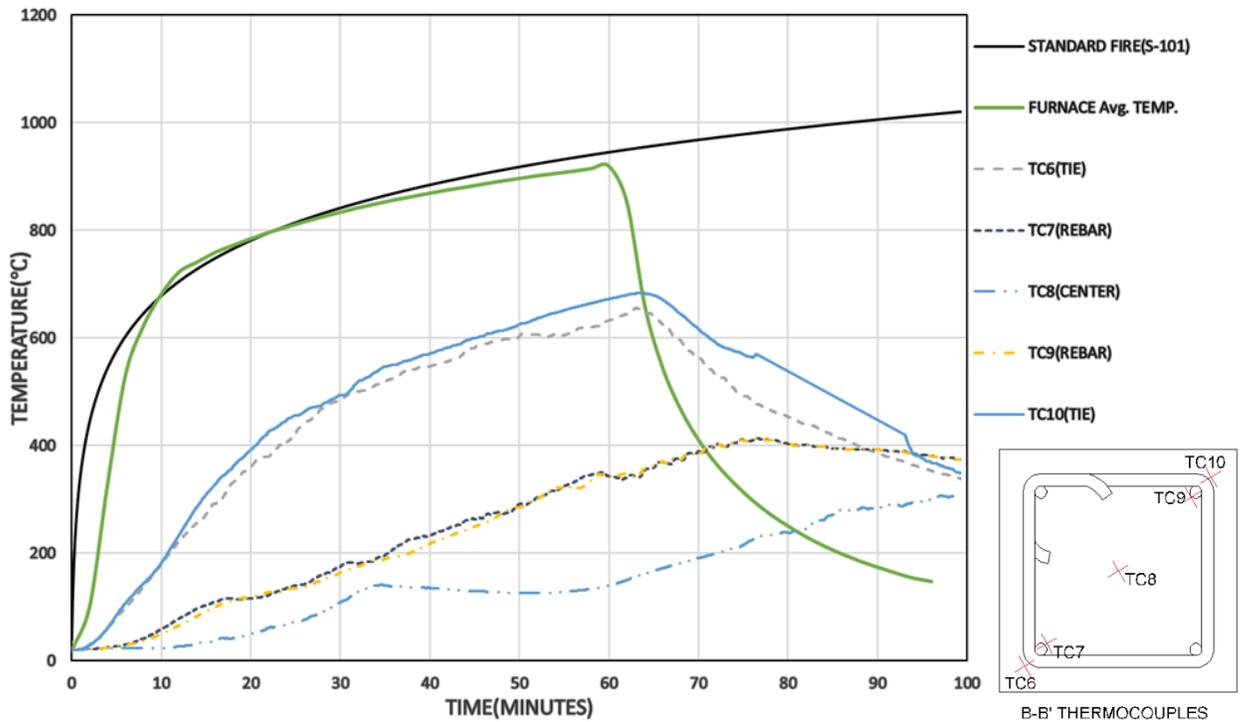
**Figure 4.5.** Time-temperature curves for a 1-hour fire duration test
(developed using thermocouples at section B-B')
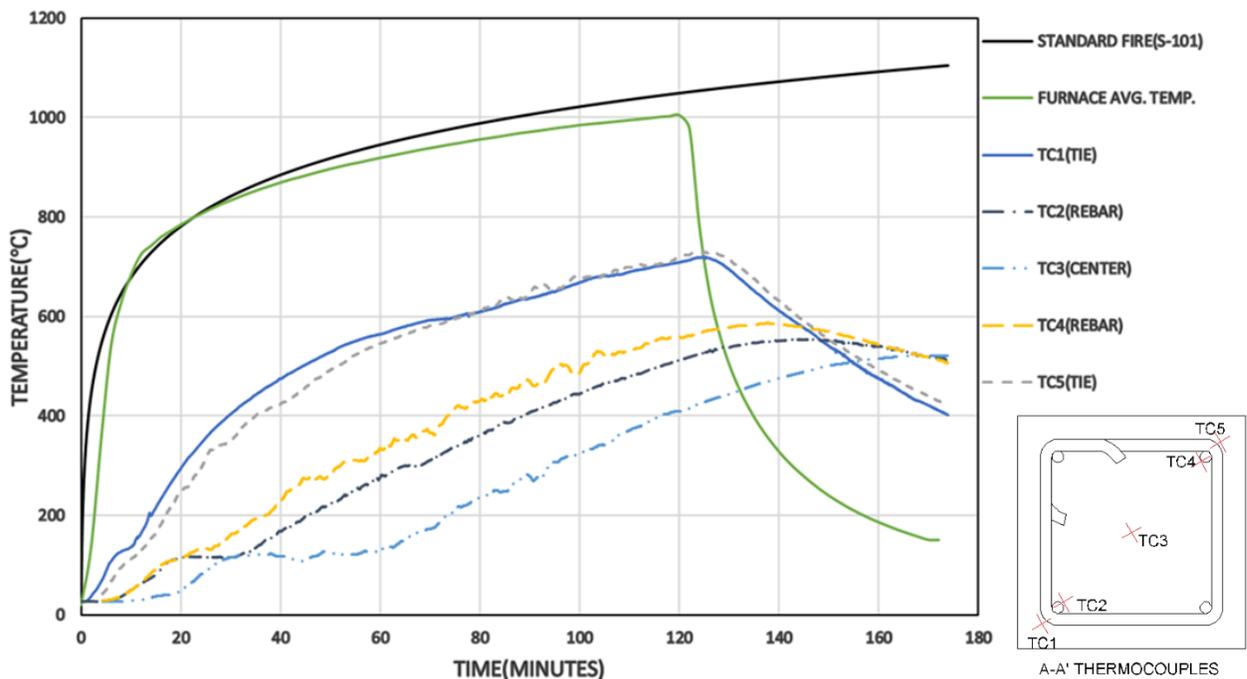


**Figure 4.6.** Time-temperature curves for a 2-hour fire duration test
(developed using thermocouples at section A-A')
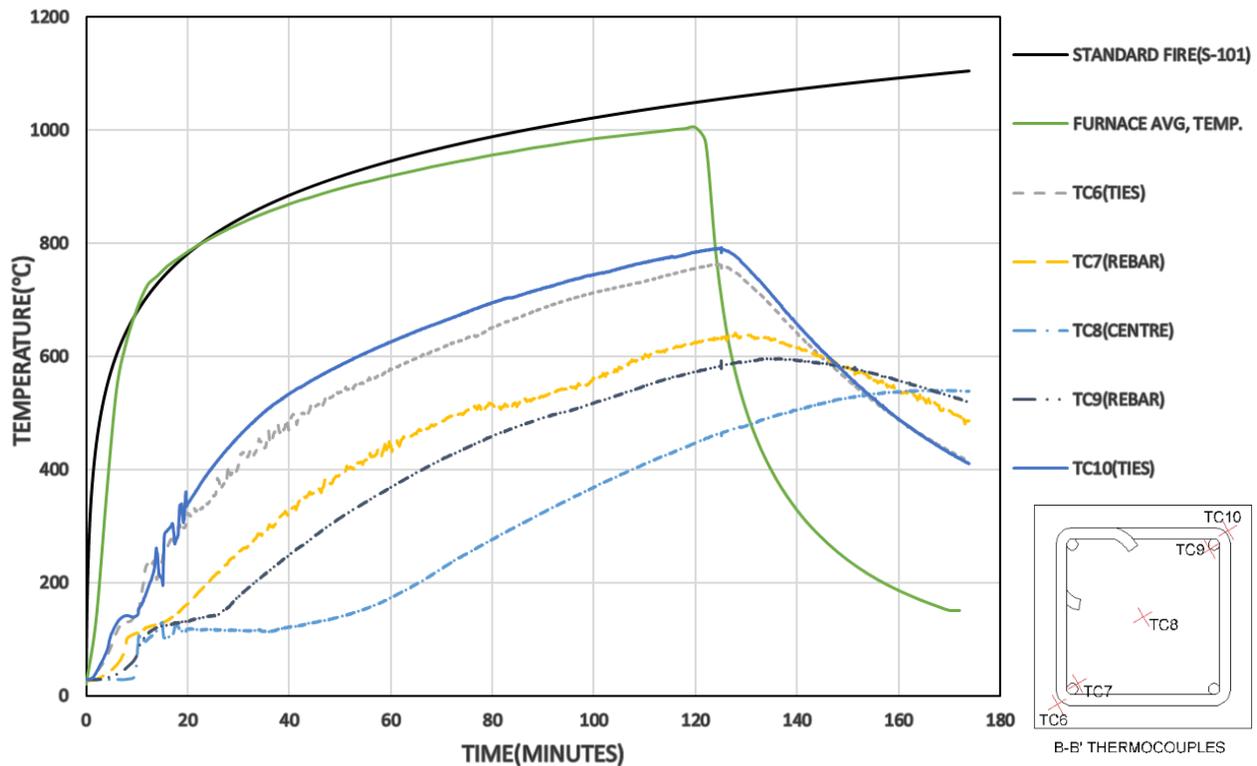
**Figure 4.7.** Time-temperature curves for a 2-hour fire duration test
(developed using thermocouples at section B-B')

The locations of thermocouples are also shown within each Figure. The fire temperatures rose rapidly within few minutes, whereas the temperatures measured by the thermocouples implemented inside the column specimens increased in a much lower rate. In all time-temperature curves, the thermal measurements of the thermocouples attached to the outer surface of the ties and the longitudinal rebar at the two opposite corners of the column cross sections have good agreements with a maximum variation of less than 50°C, which indicates that the thermal propagation through each face of the column specimen is uniform. Until five minutes of the start of the fire test, the temperatures measured inside the columns were almost at ambient temperature; however, the temperatures measured by the more outer thermocouples started to rise more rapidly than those measured by the thermocouples installed at the center of the column's cross section, and within 20 minutes, the maximum temperature measured was 300°C at the outer surface of the ties. At the same time, the temperatures measured by the thermocouples attached to the outer surface of the steel rebar reached around only 150°C. The thermal readings were much lower at

the center of the column's cross sections, as the temperatures were less than 100℃. This thermal measurement trend can be seen throughout the heating phase and it clearly indicates the lower thermal conductivity of concrete. A temperature plateau started to appear shortly before 20 minutes of the fire start in the 1-hour fire tests  and before 40 minutes in the 2-hour fire tests. This can be attributed to the latent heat consumed by free capillary water presented in the column specimens. As the pore water evaporated, temperatures inside the concrete columns started to increase with fire temperatures increase. From Figure 4.4, the maximum temperatures measured at the tie outer surface, rebar outer surface, and column center were 600℃, 350℃, and 150℃, respectively. Similarly, from Figure 4.5, the respective temperatures were 650℃, 350℃ and 150℃, respectively. The temperature profiles of both section locations (A-A' and B-B') followed very similar pattern. As shown in Figures 4.4 and 4.5, the fire tests were stopped exactly at 1 hour duration; however, the thermocouples measurements continued to read for enough time until the cooling down phase was completed. From the cooling down phase, it can be noticed that the peak temperature was attained after about 5 minutes measured by TC-1 and TC-5, and after about 15 minutes for TC-2 and TC-4 after the fire test was terminated. Similarly, the temperatures measured by TC-6, TC-7, TC-9 and TC10 increased for few minutes until reaching their peak values after terminating the fire test. It was also noticed that the temperatures measured by thermocouple TC-3 and TC-8 installed at the center of the column cross sections continued to increase for longer time compared to those thermocouples installed closer to the column's surfaces after terminating the fire tests. This can be attributed to the latent heating or thermal inertia of concrete. Meanwhile, the temperature readings of the outer thermocouples started to decrease as soon as the fire started to decay. It was also noticed that the average furnace temperatures were in very good agreement with the temperatures of CAN/ULC-S101 standard fire.

The peak average temperature readings in Figure 4.6 of the 2-hour fire tests were 700℃, 550℃, and 400℃ at the tie outer surface, rebar outer surface, and column center, respectively. Similarly, from Figure 4.7, the respective temperatures were 750℃, 600℃, and 450℃, respectively. Since the gas burners are located near the floor of the furnace, that might be the reason for the slight increase of the temperatures measured at the bottom section (B-B') compared to those measured at the top section (A-A') of all columns. The peak temperatures measured at the tie outer surface, rebar outer surface, and column center in the 2-hour fire exposed columns were higher by nearly 100℃ than those of the columns subjected to 1-hour fire exposure.

### 4.2.2 Structural Response

The main parameters investigated in this study were fire duration and load intensity ratio. During fire exposure visual observations were possible only through the furnace viewports. However, after each fire-tested column cooled down, the furnace door was removed and the columns were visually inspected to evaluate the cracks formation and spalling, Figure 4.8. Cracks and spalling mainly occurred due to the excessive pore water pressure at high temperatures. Due to the excessive tensile stresses developed by pore pressure inside the concrete, the concrete cover could break off and resulted in loss of the column's cross sections and overall strength. However, since the columns tested in this study were stored indoor in a dry environment for about 6 months, the moisture content was very limited, and thus the cracks and spalling were minor.



**Figure 4.8.** Column specimens subjected to 2-hour fire exposure and 60% load ratio,
(a) spalling from one side; (b) crack pattern

The extent of spalling and cracking were more pronounced in the columns subjected to higher load ratio since the column experience more stresses at increased loading conditions. Also, the vertical and horizontal cracks were more noticeable near the top end of the columns where the loading point was. In addition, it was noticed in all columns underwent 2-hour fire exposure that the column's four corner edges were damaged as a result of the excessive heat and consequent degradation of the strength of concrete. Figure 4.9 shows two identical columns underwent 2-hour fire exposure, and the damage occurred in the column four corners.
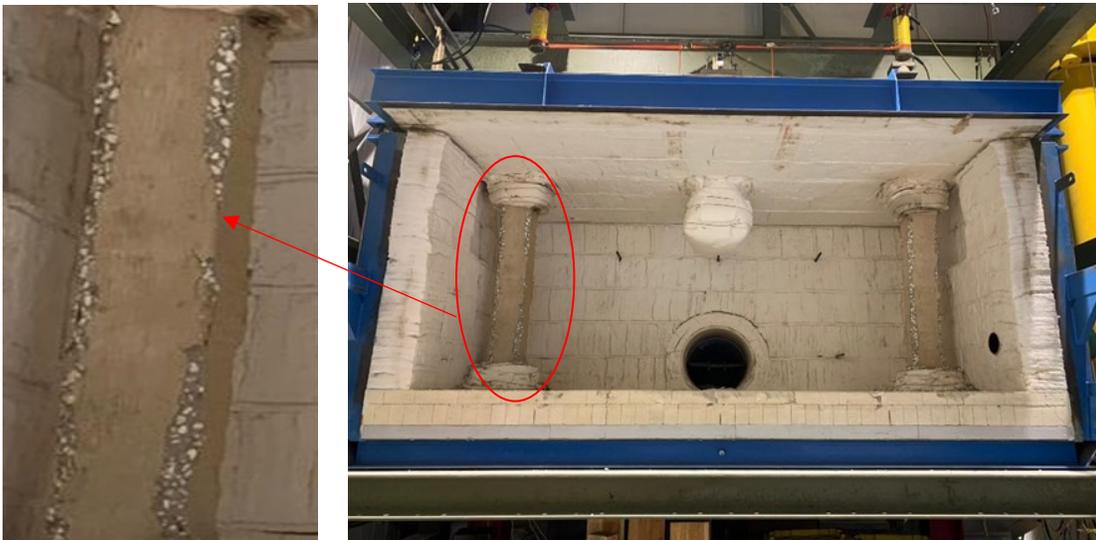


**Figure 4.9.** Column specimens underwent 2-hour fire exposure with its corners damaged

# Chapter 5   Residual Strength Tests Results and Discussion

## 5.1     General

The information regarding residual structural capacity of fire damaged concrete structures that have undergone large deformations, cracks or spalling is limited. The prediction of residual capacity after fire exposure is unreliable. Hence, fire testing of reasonably scaled concrete columns is necessary to evaluate the actual residual compressive strength of such columns. The residual strength of RC columns is affected by several parameters including fire severity, load level, material properties, restraint conditions, thermally-induced bond degradation, cooling conditions and more. Unfortunately, there are not enough studies available on the effect of these factors that are all interdependent, which makes it more complicated to predict the actual residual strength of such fire-damaged columns. Instead of demolishing and reconstructing fire exposed structures, engineered methods of retrofitting can be effectively utilized, which is a time saving and efficient technique. In order to adopt such methods, it is important to have sound knowledge of the residual compressive strength of fire-damaged structures.

All eight fire-damaged columns tested in this study, after cooled down to ambient temperature, were loaded to failure to determine their residual compressive strength. The load-deformation relationships, crack propagation and failure modes were monitored during residual strength tests. A few localized structural failures were seen in all column specimens after fire exposure due to thermal creep, localized or global plastic deformation, thermal expansion, restraint conditions, etc. Figure 5.1 shows a general residual stress test setup.

**Figure 5.1.** A general column residual strength test setup

## 5.2 Structural Response

During residual strength testing, the axial deformations of the columns were measured using draw-wire displacement transducer built in the UTM. In all fire-damaged columns, splitting off the concrete cover occurred, and the failure was mainly due to crushing. During loading, widening of the developed cracks followed by spalling of concrete. Eventually, vertical cracks were observed almost at the mid height of the columns and concrete started to burst out, and then steel rebar buckled as the column attained failure. The failure of all columns was more of explosive in nature. The failure has occurred when the column could no longer withstood the applied load. The maximum load recorded was considered as the residual compressive strength of the tested specimens. Axial deformation pattern was monitored during the residual strength testing. Figure 5.2 shows the residual load-deformation relationships of two of the fire-damaged column specimens; one exposed to 1-hour fire duration, while the other exposed to 2-hour fire duration, and both subjected to 40% load ratio.
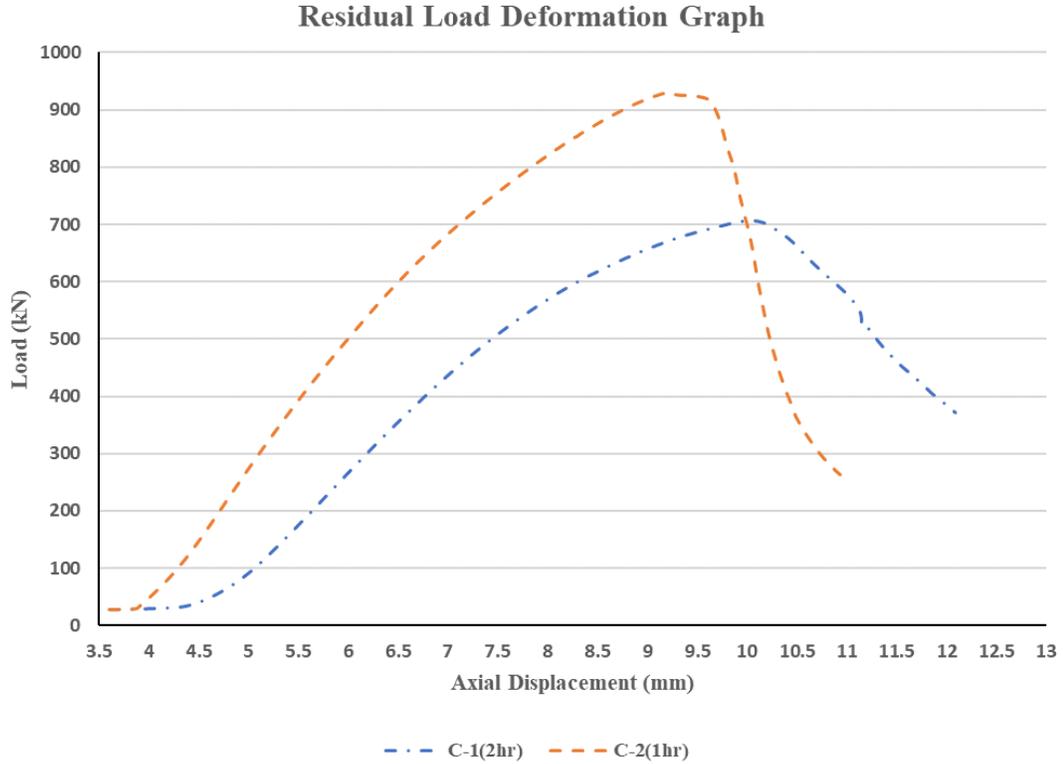
**Figure 5.2** Residual load-deformation relationships for two column specimens exposed to 1- and 2-hour fire duration, and both subjected to 40% load ratio

From Figure 5.2, it can be noticed that during initial stage (linear elastic stage), both columns followed linear trend until the yielding occurred. The column that was exposed to 1-hour fire duration experienced less increments of the axial deformations with load compared to those experienced by the 2-hour fire exposed column. During the second stage (inelastic hardening), the concrete cover began to bulge outward and peel off. Also, the maximum axial deformation occurred in C-1 and C-2 were 10.0 mm and 9.25mm under failure loads of 706.6 kN and 963.2 kN, respectively. After reaching the failure load in each column, the axial deformations continued to gradually increase while applied load started to drop. The residual compressive load carrying capacity for the 1-hour fire exposed column was greater than that of the 2-hour fire exposed column, both subjected to 40% load ratio, by about 36%. This can be attributed to the fact that the 2-hour fire exposed column experienced more thermally induced damage. When comparing post-fire deformation and ambient temperature deformation, the axial displacements exhibited during residual strength tests were greater than those exhibited during ambient temperature compressive tests. This can be related to the changes occurred to the mechanical and thermal properties of the

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concrete as a result of fire exposure, e.g., reduction in the column cross section due to spalling, internal and external cracks, and plastic centroid movement.

The fire-damaged column specimens which were subjected to 20% load ratio for 1-hour fire duration failed at an average compressive load of 1048.8 kN which reflects a residual compressive strength of about 55% of its ultimate load capacity. Residual strength reduced to an average failure load of 963.2 kN for the fire-damaged column specimens exposed to same fire duration but had double the load ratio of 40%. Whereas for the column specimens exposed to double the fire duration (2-hours instead of 1-hour), significant drops in their residual strength were observed, as the third and fourth fire damaged column specimens subjected to 20% and 40% load ratios failed at an average load of only 727.3 kN and 706.6 kN, respectively. This indicates that the fire duration had more influence than the variation of the load ratio on the residual compressive strength of the fire damaged columns; as the residual compressive strength of the columns that underwent 2-hour fire tests reduced to only 37% of their ultimate load capacity.

A summary of the results from the residual strength tests of all eight fire damaged column specimens are presented in Table 5.1.

**Table 5.1.** Residual compressive strength test results

| Column designation | Average ambient ultimate failure load (kN) | Fire duration (hours) | Applied load during fire (kN) | Residual compressive strength | |
|---|---|---|---|---|---|
| | | | | (kN) | (%) of ambient ultimate capacity |
| C1(A)-20% | 1879.3 | 1.0 | 180.0 | 1116.0 | 55.8% |
| C1(B)-20% | | | | 981.7 | |
| C2(A)-40% | | | 360.0 | 996.4 | 51.2% |
| C2(B)-40% | | | | 930.0 | |
| C3(A)-20% | | 2.0 | 180.0 | 709.0 | 38.7% |
| C3(B)-20% | | | | 745.5 | |
| C4(A)-40% | | | 360.0 | 705.9 | 37.6% |
| C4(B)-40% | | | | 707.2 | |

Using the obtained experimental results, it is possible to compare the effect of fire duration and load ratio on the residual compressive strength of the different tested column specimens. The combined effect of elevated temperatures of fire and the applied structural load has significant influence on the residual compressive strength of RC columns. It caused the compressive strength to reduce to almost half of its ambient ultimate magnitude for the columns exposed to 1-hour fire duration, and to almost one-third of its ambient ultimate magnitude for the columns exposed to 2-hour fire duration. However, the residual compressive strengths of all fire-damaged columns were greater than the ultimate design compressive strength of each respective column.

**5.3 Discussion**

In this experimental research the specimens taken are short columns with 200 x 200 mm cross-section and 1500mm height. The maximum design capacity was calculated as 940 kN and from compressive strength experiment, the ultimate strength of column was obtained as 1879.3 kN. The difference between design value and experimental value may be due to the increased compressive strength of concrete as aging, material reduction factors considered for designing or due to changes in yield strength of steel from theoretical values. In the similar experimental study conducted by Kodur *et al*. (2017), had considered similar cross-section of columns (203 x203 mm) but long columns of 3.35m high. The tie spacing (200 mm center to center) used was same in both studies. The increased confinement in our specimen might have affected the increased axial capacity. From literature, the nominal capacity was reduced to 33.6% and 29% after a fire duration of 90 minutes and 120 minutes. But in this current research, the residual capacity was dropped to almost 50% and 30% of ultimate capacity after being subjected to 1 hour and 2-hour fire durations. This clarifies that the slender columns are severely affected by fire exposure.

**5.3.1    Effect of load ratio**

During fire exposure, the columns were subjected to two different load ratios, 20% and 40% of the column's ultimate design capacity. The load was applied 30 minutes before the start of the fire test and was maintained throughout the entire duration of the test. In real fire scenario, none of the building structures are usually subjected to its 100% of load carrying capacity. Furthermore, the evacuation of buildings in fire incident leaves the building with much less live load existing. In order to imply this real condition, load intensity ratio was taken as a study parameter.
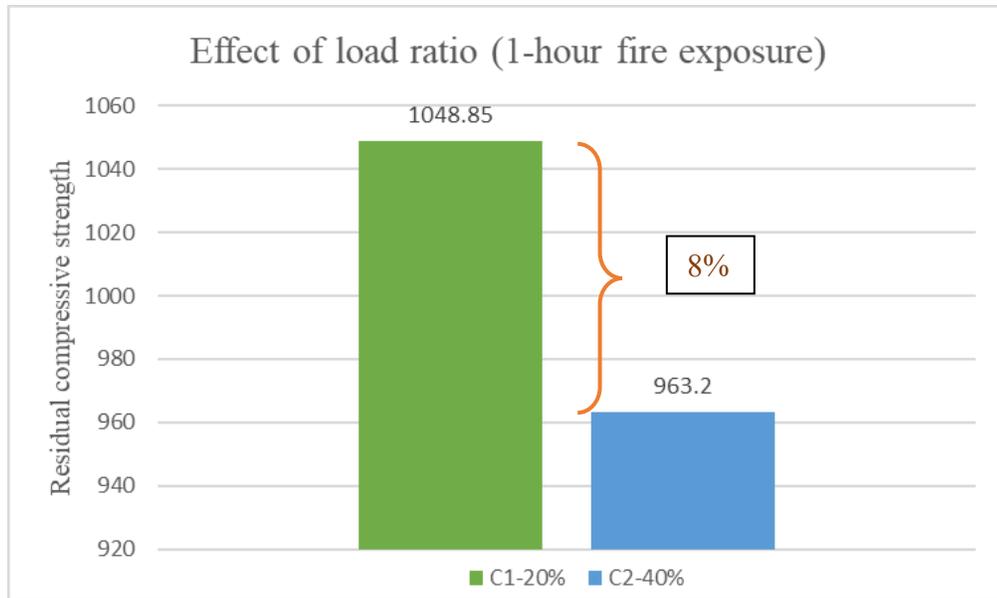
**Figure 5.3** Residual compressive strength of columns subjected to different load ratios (1-hour)
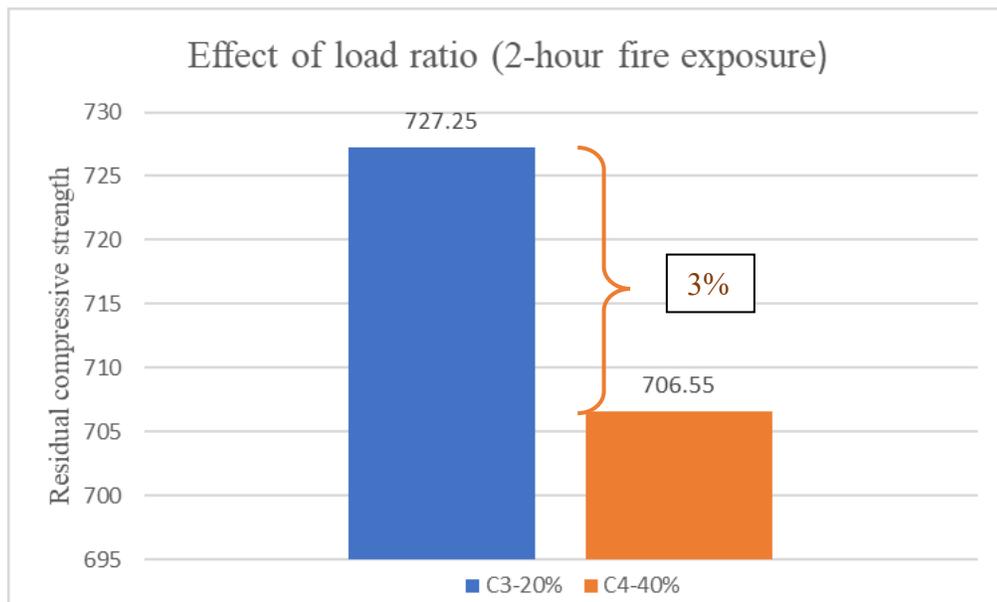


**Figure 5.4** Residual compressive strength of columns subjected to different load ratios (2-hour)

Figure 5.3 and 5.4 show the variation of residual strength when the columns were subjected to different load ratios. During residual strength tests, it was observed that the fire-damaged columns that were subjected to higher load ratio (40% of their ultimate design load capacity) failed at less residual compressive loads than those in the columns subjected to less load ratio. Although the

temperatures propagation inside the column cross sections wasn't influenced by the ratio of the applied load by any means, at higher load ratio, the columns were more restrained against thermal expansion during fire exposure, and thus more thermally induced damage occurred in such columns. Also, although the effect of the load ratio on the residual compressive strength of the fire-damaged columns was much less compared to the effect of the fire duration, still there was a measurable compressive strength reduction due to the application of higher load ratio in the percentage of about 8% and 3% when the load ratio was doubled (40% instead of 20%) for 1- and 2-hour fire durations, respectively.

### 5.3.2 Effect of fire duration

Generally, concrete buildings subjected to real fire perform well and its complete structural failure is rare to occur. For relatively short fire exposure of 1 hour or less, a concrete building often requires a renovation, and in certain cases retrofitting or rehabilitation procedure might be required, dependent on the actual residual strength of the most affected structural components in the building. There are several studies have been conducted to analyze the performance of post-fire concrete buildings. One important factor taken into account in such studies was the severity of fire. The type of fire and its duration have important role in the fire resistance of the different structural components of a concrete structure as well as its residual strengths. Building fires, or compartment fires, develop over three phases: growth phase, developed fire (steady) phase, and decay phase. Each phase depends on the available fuel load and ventilations. The growth of fire is dependent on time as well as the availability of sufficient fuel. Afterwards, and once the fire is fully developed and went into steady burning stage, the time extension of this stage has significant impact on the residual strength of the building's structural components exposed to the elevated temperatures of the fire. The time to failure is considered as the fire resistance of the structural components.
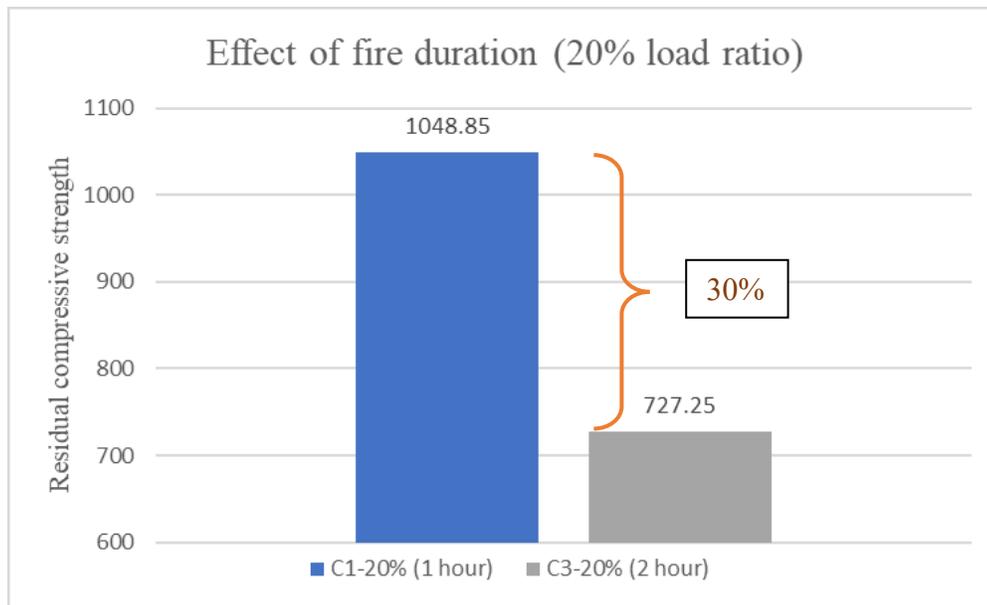
**Figure 5.5** Residual compressive strength of columns subjected to

different fire durations (20% load ratio)
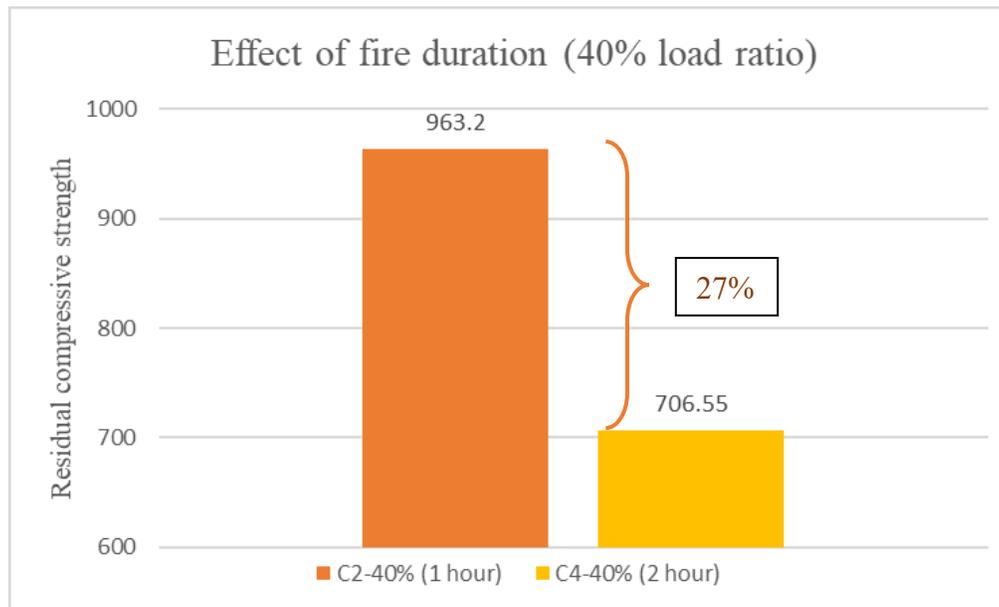


**Figure 5.6** Residual compressive strength of columns subjected to

different fire durations (40% load ratio)

From Figure 5.5 and 5.6, when comparing the residual compressive strengths of 1- and 2-hour fire exposed columns subjected to 20% and 40% load ratios, there were significant reductions in the percentage of 30% and 27%, respectively. This indicates that the fire duration has very similar

influence on the reduction of the residual compressive strength of fire-damaged columns regardless of the load ratio applied on the columns.

### 5.3.3 Failure modes and crack patterns

During residual compressive strength tests, vertical cracks were formed while reaching ultimate load capacity of the columns. In all fire-damaged columns, the failure occurred at almost the one-third height of column from the top end where the applied load was. At the initial stage, cracks began to form along the middle half-height of the columns and then the columns started to bulge. Concrete chunks from the columns cover on the four sides started to breakoff and fall apart. The failure mode of all columns from the first stage of loading was explosive in nature. Afterwards, concrete burst outward resulting in the reinforcing steel rebar to become exposed and then exhibited visible buckling. Figure 5.3 shows the failure of the different four fire-damaged RC column specimens upon completion of the residual strength tests.



|  (a) | (b) | (c) | (d) |
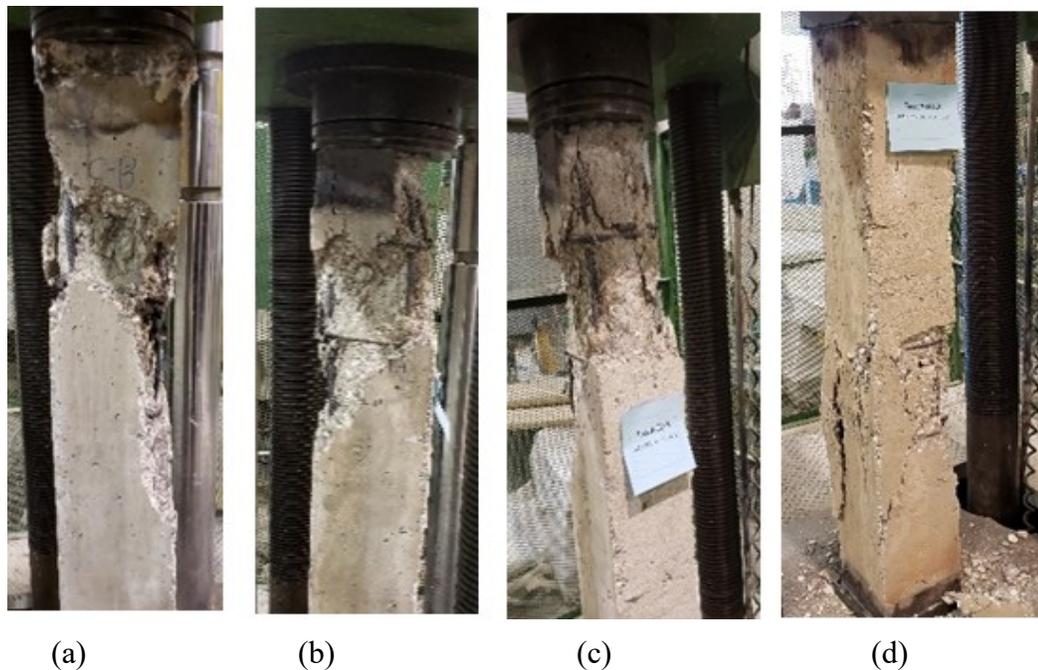
**Figure 5.7.** Failure of fire damaged columns after being loaded, (a) column of 1-hour fire duration and 20% load ratio; (b) column of 1-hour fire duration and 40% load ratio; (c) column of 2-hour fire duration and 20% load ratio; (d) column of 2-hour fire duration and 40% load ratio.

# Chapter 6   Conclusions and Recommendations for Future Work

## 6.1    General

Concrete is widely used in the construction of buildings, bridges and different type of structures around the world, and it is considered as the most fire-resisting construction material. Despite the several studies that investigated the fire behavior of concrete as a structural material, it appears that the fire resistance of concrete is not always perfectly predictable due to several reasons and factors that affect the mechanical and thermal properties of concrete at elevated temperatures.

Over the past few decades, several RC structural elements and assemblies were tested for fire resistance ratings under different conditions. According to the prescriptive-based design approach, the goal is to achieve code specified fire resistance rating, where the clauses of a building code explain the type of occupancies and the corresponding fire resistance rating required for certain structural components. For common RC buildings, general information, e.g. minimum cross-sectional dimensions, cover of reinforcement, etc., are listed in different building codes to achieve specific fire resistance ratings, as required. However, nowadays, building fire safety and structural engineering industries started to replace the performance-based design over prescriptive-based design, since the performance-based design allows the designers the ability to address the unique features of each building and find cost-effective solutions and design alternatives to meet the required fire resistance rating for the building's different structural components.

Nevertheless, there are many concrete buildings remained unused after fire incidents. This is mainly due to the unawareness of the actual load-bearing capacity of the different structural components in such fire-damaged buildings. It would be very practical and economically wise if structural condition assessment being carried out for such fire-damaged buildings so they can be reused after performing proper retrofitting/rehabilitation measures instead of demolishing such buildings. Therefore, it is very crucial to properly assess the residual strength of the different structural components of fire exposed buildings.

In concrete buildings, columns are the main loadbearing elements that transfer the loads to the foundations. There are studies, both experimental and numerical, available in literature that investigated the fire resistance of RC columns under different fire scenarios. According to those studies, it was concluded that it is rare that a concrete building completely collapses after even

prolonged fire incident. Unfortunately, there is only limited information available on the residual strength of RC columns after being exposed to fire, which is imperious for performing effective retrofitting works for such fire-damaged columns. Determining the residual strength of fire-damaged RC columns is a complex task, and the current approach mainly utilizes some imperial equations with temperature-dependent strength reduction factors.

Accordingly, it was important to carry out the unique experimental study presented in this thesis to allow for better understanding of the behavior of reinforced concrete short columns exposed to varying fire durations and subjected to different load intensity ratios, as well as to experimentally determine the residual compressive strength of such fire-damaged columns.

## 6.2 Conclusions

Based on the results obtained from the experimental study presented in this thesis to determine the residual compressive strength of eight reinforced concrete short columns that were subjected to two different load ratios for two different standard fire durations, the following conclusions can be drawn;

- Fire exposure considerably affects the compressive strength of all reinforced concrete columns tested in this study. Also, even though concrete has low thermal conductivity, once the columns exposed to high temperatures, they never regain their original compressive strength;

- Even though concrete possesses superior thermal properties, the peak rebar temperature reaches up to 350°C and 600°C after 1-hour and 2-hour fire exposure duration, respectively. Also, during the heating phase of the fire, the maximum temperature attained at the center of the column was lesser than all other readings.

- During the cooling down phase, the temperatures measured at the center of the concrete column cross sections kept increasing for a few minutes after terminating the fire test. Whereas, the outer portion of concrete cross section near the column's surfaces started to cool down immediately after the termination of the fire test and faster than the core portion of the concrete columns, mainly due to the latent heating inside the concrete, which also caused the concrete to experience unrecoverable strength lose ;

- All column specimens survived the 1- and 2-hour standard fire exposure with minimum cracks and spalling developed. However, the average residual compressive strengths of fire

damaged column specimens subjected to 20% and 40% load ratios for 1-hour fire duration were about 55.8% and 51.2% of the column's ultimate load capacity at ambient temperature, respectively. Whereas the average residual compressive strength of fire damaged column specimens subjected to 20% and 40% load ratios but for 2-hour fire duration were about 38.7 % and 37.6% of the column's ultimate load capacity at ambient temperature, respectively;

- During fire testing, the combined effect of fire and mechanical loading resulted in some spalling and cracking. This caused reduction of the column's cross section, strength reduction, and chances of some eccentricity. Also, spalling and cracks were more noticeable in the specimens that underwent longer fire duration and subjected to higher load ratio;

- The residual compressive strength test results clarified that fire had severely affected the load-bearing capacity of RC columns. Also, the percentage of the column's ultimate compressive strength had dropped to almost half and one-third after 1- and 2-hour fire exposure durations, respectively. However, the residual compressive strength values of all fire-damaged columns passed the column's ultimate design capacity at ambient temperature;

- Although the effect of the load ratio on the residual compressive strength of the fire-damaged columns was much less compared to the effect of the fire duration, still there was a measurable compressive strength reduction due to the application of higher load ratio in the percentage of about 8% and 3% when the load ratio was doubled (40% instead of 20%) for 1- and 2-hour fire durations, respectively;

- The 1- and 2-hour fire exposed columns subjected to 20% and 40% load ratios experienced significant reductions in its residual compressive strength in the percentage of 30% and 27%, respectively. This indicates that the fire duration has very similar influence on the reduction of the residual compressive strength of fire-damaged columns regardless of the load ratio applied on the columns.

## 6.3    Recommendations for Future Work

Although the primary goal of the experimental study presented in this thesis was to determine the residual compressive strength of reinforced concrete short column exposed to different fire durations and load intensity ratios, there are other inter-dependent factors which may affect the residual strength of post-fire reinforced concrete columns. Addressing and evaluating such factors would help to get a vivid understanding of the residual compressive strength of such fire damaged columns.

The followings are a few recommendations for future work to be conducted in this area;

- Based on the unique experimental outcomes of this study and since the fire damaged reinforced concrete columns possessed considerable residual compressive strength, in particular those exposed to shorter fire duration, it is anticipated that with proper retrofitting techniques, such as FRP wrapping, the fire damaged columns can be rehabilitated to regain at least portion of their lost load bearing capacities. Accordingly, the residual compressive strength data obtained from this study can be used to adopt optimal retrofitting strategies for such fire damaged concrete columns;

- The temperature profile data and residual strength response data obtained from the experimental program can be utilized to develop and validate finite element models of such fire damaged columns. Such models can be used to conduct parametric studies to investigate the effects of factors such as various fire scenarios, load levels, and column's different cross-sectional details on the residual compressive strength of the studied fire damaged columns. This can also help to develop simplified design approaches to evaluate the residual strength of post-fire concrete columns;

- Since the availability of experimental data for residual strength of fire exposed columns is very limited, it is also beneficial to expand the spectrum of the experimental program that the study presented herein was part of it to include testing of similar column specimens but to investigate the effects of other parameters such as the grade of concrete and steel reinforcement, end restraint conditions, cross section shape, concrete cover, column's slenderness ratio, and various degrees of confinement. Furthermore, various cooling conditions can also be very beneficial to explore.

# Appendix A: Load Carrying Capacity Calculations for Tested Columns at Ambient Temperature

The design of RC column test specimens has been done according to CSA-A23.3-14.

**Table A-1.** Parameters for the design specimen

| Parameter | Value |
|---|---|
| Cross-sectional dimension | 200 x 200 mm |
| Longitudinal steel reinforcement | 4x 10M rebar |
| Stirrups | 10M rebar at 200mm c/c |
| Yield strength of steel | 400 MPa |
| Concrete Cover | 30 mm |
| Column Height | 1500 mm |

**LOAD CALCULATION:**
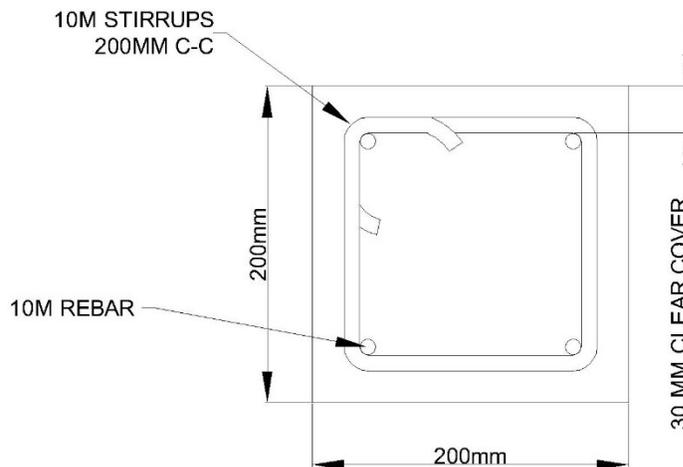


**Figure A.1** Column Cross-section

Slenderness ratio;

- Length of the column = 1500 mm
- Since both ends are assumed as pinned end restrained condition;
- Effective length factor, $k$ = 1

- Radius of gyration, $r = 0.3h$, where $h$ is the dimension of the column in the direction of eccentricity

$$h = 200 \text{ mm}$$

$$r = 0.3 \times 200 = 60 \text{ mm}$$

- Unbraced length of column, $l_u = 1500$ mm

$$\text{Slenderness ratio} = \frac{k\,l_u}{r}$$

$$= \frac{(1.0)\,(1500)}{60}$$

$= 25$, Since it is small, slenderness may therefore neglect.

Equation A.1.　　　　Factored axial load resistance, $P_{ro} = \alpha_1 \emptyset_c f_c{'} (A_g - A_{st}) + \emptyset_s f_y A_{st}$

(A23.3 Eq.10.10)

Where,　　　　$P_{ro}$ = Factored axial load resistance

　　　　　　　$A_g$ = Gross area of column cross section

　　　　　　　$A_{st}$ = Total area of longitudinal reinforcement

$A_g = 200\text{x}200 = 40000 \text{ mm}^2$

$A_{st} = 4\text{x}100 = 400 \text{ mm}^2$

$f_c{'} = 50$ MPa (characteristic compressive strength of concrete on test day)

$P_{ro} = 0.80\text{x}0.65\text{x}50\text{x} (40000 - 400) + 0.85\text{x}400\text{x}400$

　　$= 1165600 \text{ N} = 1165.6 \text{ kN}$

A23.3 Cl.10.10.4 accounts for the effect of unanticipated or accidental moments in concentrically loaded columns by prescribing a reduction in the column axial load resistance.

Equation A.2　　　　For tied columns;　　$P_{rmax} = 0.80\,P_{ro}$ (A23.3 Eq.10.9)

$P_{rmax} = 0.80\text{x } 1165.6$

　　$= 932.5 \text{ kN}$

Therefore, the maximum axial loads this column can carry according to CSA A23.3 is 603kN.

For design purposes, $P_{rmax}$ value can be calculated using simplified equation;

$P_{rmax} = 0.8(\alpha_1 \emptyset_c f_c{'} A_g + \emptyset_s f_y A_{st})$

　　$= 940.8 \text{ kN}$

# Appendix B: Concrete Mechanical Properties at Elevated Temperatures
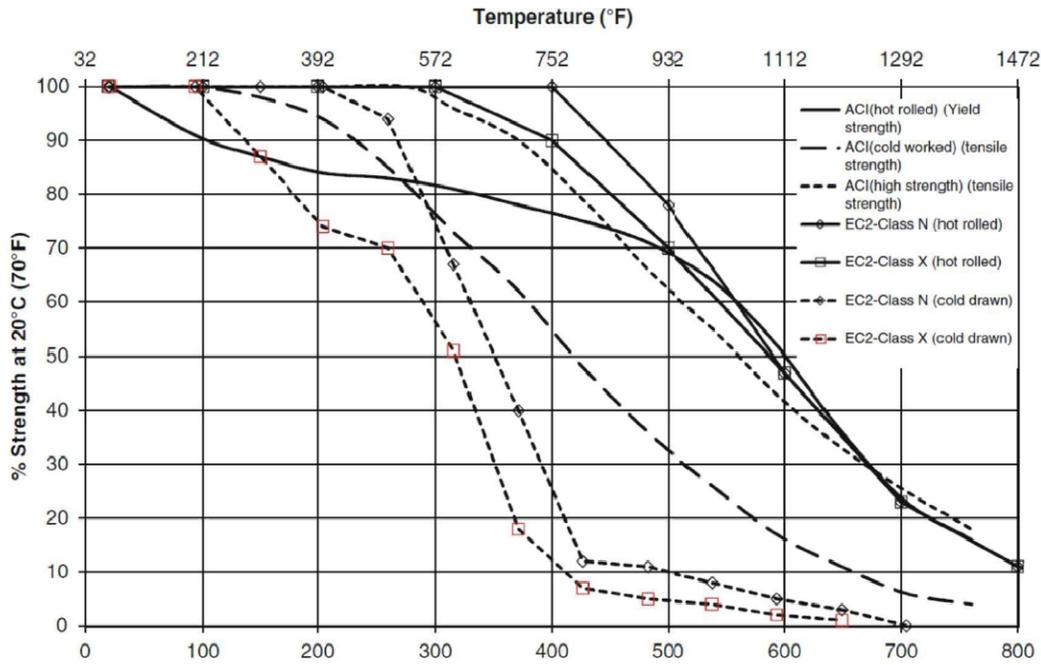


**Figure B-1.** Strength-temperature relationships for reinforcing steel (Adapted from Eurocode 2)
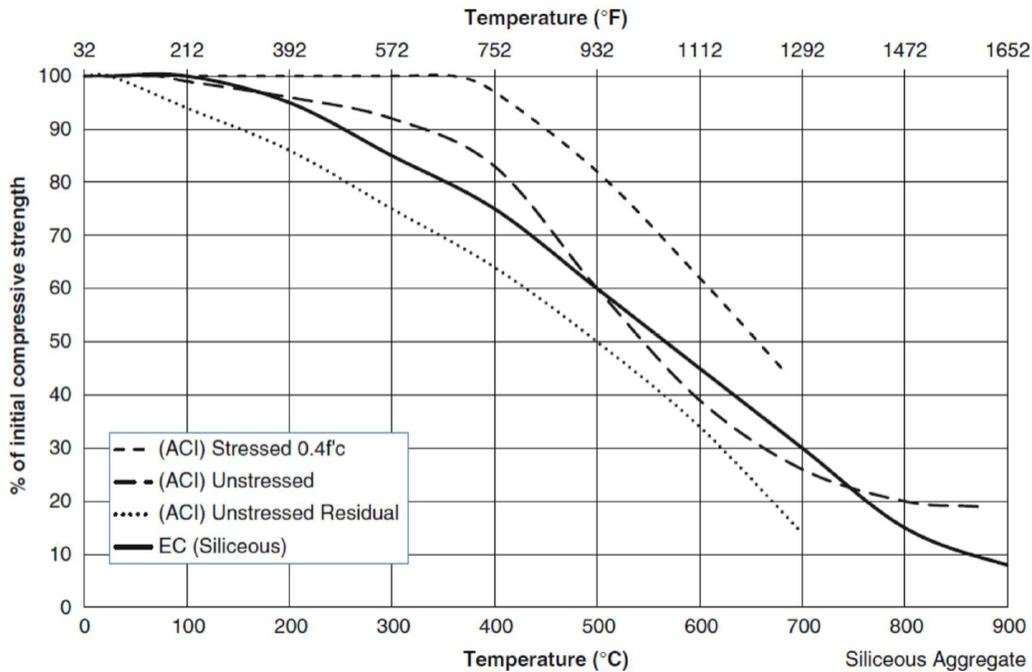


**Figure B-2.** Strength-temperature relationships for Siliceous Aggregate
(Adapted from Eurocode 2)
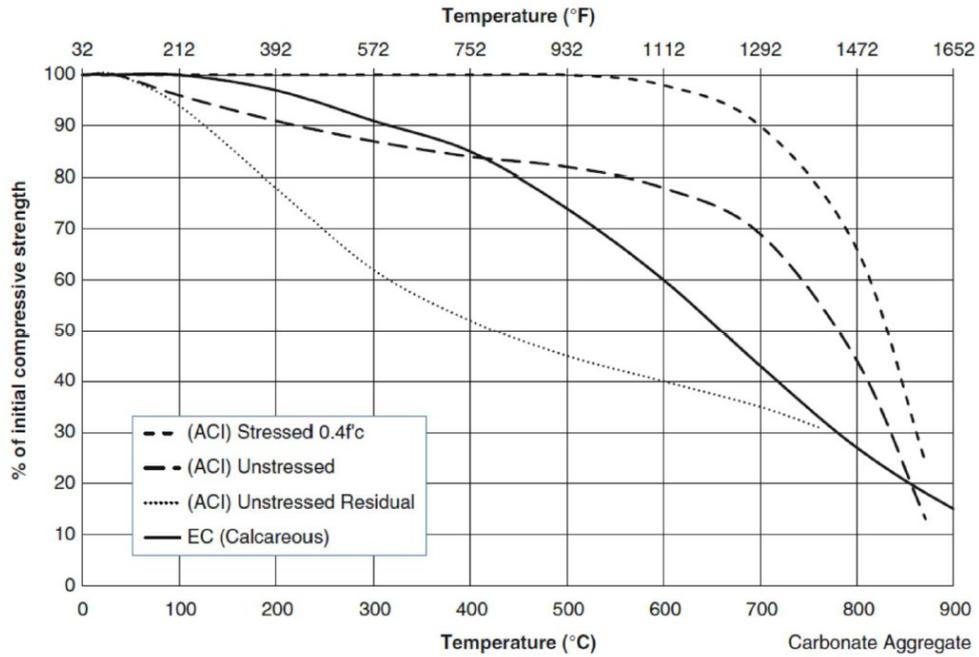
**Figure B-3.** Strength-temperature relationships for Carbonate Aggregate
(Adapted from Eurocode 2)
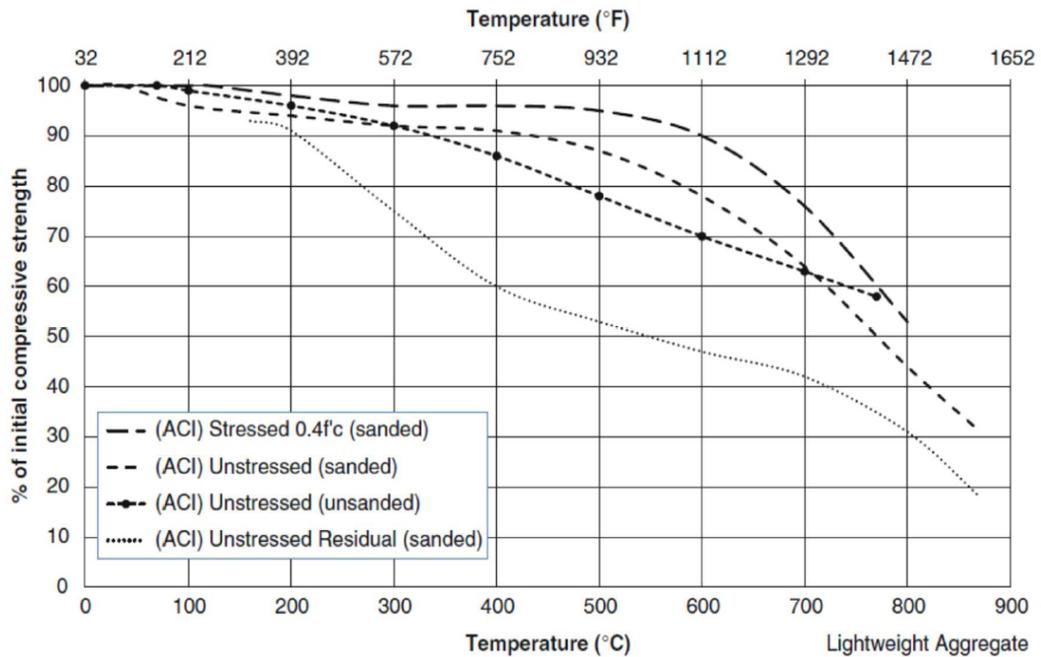


**Figure B-4.** Strength-temperature relationships for Lightweight aggregate
(Adapted from Eurocode 2)

# Appendix C: Formwork Design
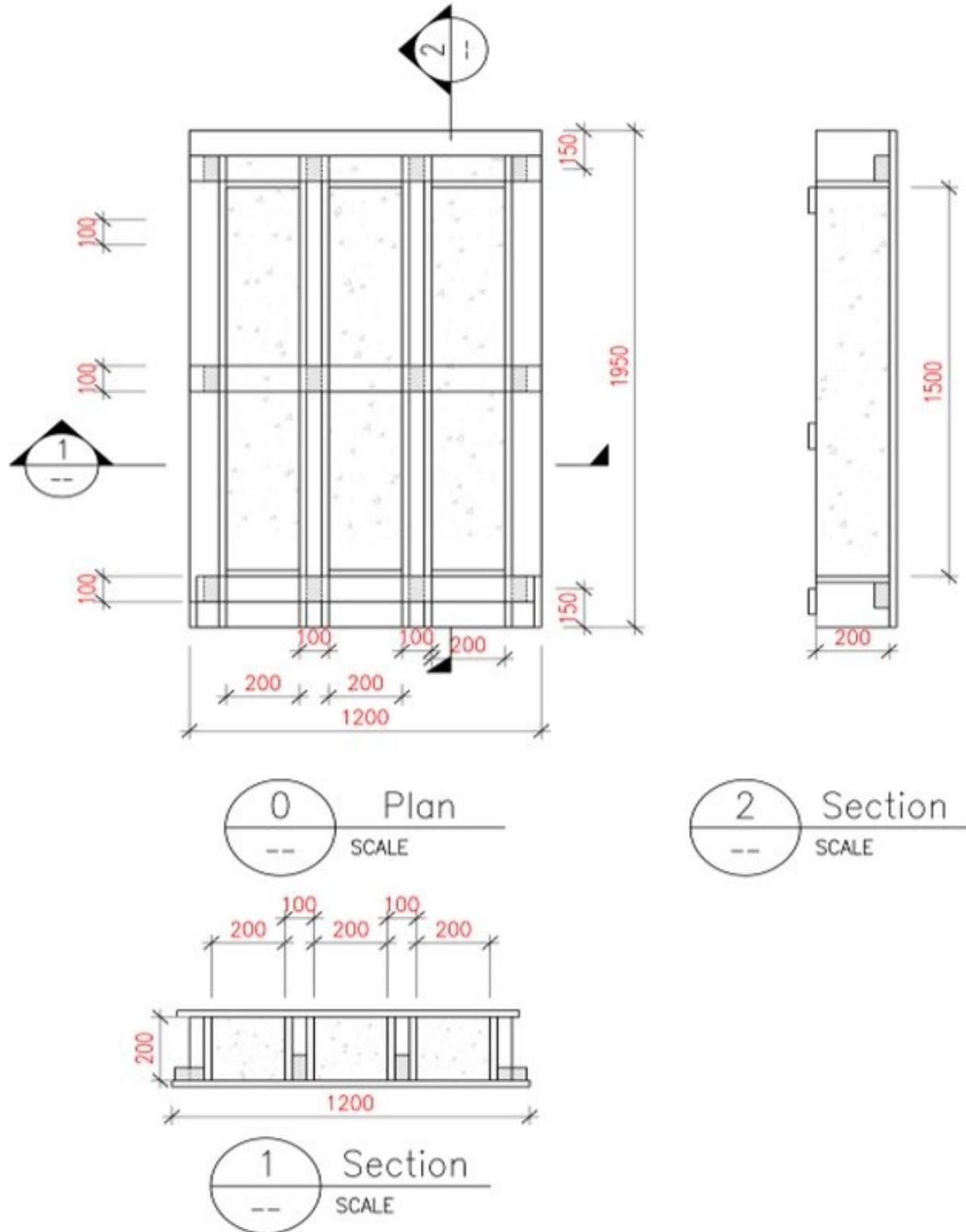


**Figure C-1.** Column specimens' formwork details

## Appendix D: Heat Transfer Analysis (Wickstrom's Method)

After 1-hour fire exposure:

Equation D.1   Standard fire temperature, $T_f = 20 + 345\log(8t + 1)$

where;

$t$ – time in minute

$$T_f = 20 + 345 \log 8(60) + 1 = 945°C$$

Equation D.2   Surface Temperature, $T_w = 20 + \eta_w (T_f - 20)$

where;

$\eta_w = 1 - 0.0616\, t_h^{-0.88}$          $t_h$ – time (hour)

$= 1 - 0.0616 \times (1)^{-0.88} = 0.938$

$$T_w = 20 + 0.938 (945 - 20)$$
$$= 887.62°C$$

Equation D.3   Concrete temperature at 'x' distance where the center of rebar is:

$T_c = 20 + [\, \eta_w (\eta_x + \eta_y - 2\, \eta_x\, \eta_y) + \eta_x\, \eta_y]\, (T_f - 20)$

$\eta_x = 0.18 \ln (t_h/x^2) - 0.81 = 0.397 = \eta_y$   ; $x = 30 + (10/2) = 35$ mm

$T_c = 579.43 \,°C = T_{steel}$

Equation D.4   Reduced yield strength:

$f_{yt} = \dfrac{f_y (720 - T_s)}{470}$

$= 119.63$ MPa

Reduced compressive strength of concrete at around 600°C, $f'_{c,\, t} = 18$ MPa (60% of the initial compressive strength)

Reduced axial load resistance $= P_{ro} = \alpha_1\, \varnothing_c\, f_{c,t}'\, (A_g - A_{st}) + \varnothing_s\, f_{y.t}\, A_{st}$

$= P_{ro} = 0.80 \times 0.65 \times 18 \times (40000 - 400) + 0.85 \times 119.63 \times 400$

$= 411330.2$ N $= 411.3$ kN.

$P_{rmax} = 0.80 \times 411.3$

$= \textbf{329.06 kN}$.

Hence, there is around 54% strength reduction after 1-hour fire exposure.

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